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MEMBERSHIP

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

PAPERS

ECONOMIC PROPORTIONS AND WEIGHTS OF MODERN HIGHWAY CANTILEVER BRIDGES

By J. A. L. Waddell,[1] M. Am. Soc. C. E.

Synopsis

During the last decade there has been quite a development in the construction of long-span cantilever highway bridges; and a number of such structures have been designed in the writer's office. As the computations have always been made and recorded systematically, it was practicable to obtain weights of steel per linear foot of structure for the various divisions of the metal, namely, trusses, floor system, lateral system, anchorages, and on piers; and with these, by some established empirical formulas of transition, it was feasible to pass from known weights per foot in any layout to the corresponding weights per foot in a different but somewhat similar layout, and, in that way, and by means of moment-area diagrams, to determine all the economic functions for this type of highway bridge.

In 1897 the writer made a similar, but much less extensive, research for single-track railway cantilever bridges, and published the results in his little treatise, "De Pontibus," now out of print, but absorbed in his larger work, "Bridge Engineering."

As was anticipated, the economic proportions proved to be somewhat different in the two types of bridges; because in the first case the structures investigated were narrow, their live loads were comparatively large, and their dead loads were small; whereas in the second case the structures investigated were generally wider, their live loads were comparatively small, and their dead loads were great, owing to the heavy highway flooring.

The investigations thus made permitted the writer to prepare diagrams of total weights of metal per square foot of floor for modern, highway cantilever bridges up to main openings of 1 200 ft., beyond which it is not worth while to go, because, under all but very exceptional conditions, a suspension highway bridge is cheaper for that length of span.

Note.—Written discussion on this paper will be closed in May, 1932, *Proceedings*.

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To round out the paper there have been added diagrams of total weights of metal per square foot of floor for simple-truss spans, in both silicon steel and carbon steel. It is shown that silicon steel for cantilever bridges is always cheaper than carbon steel, and that for present unit prices the dividing length in simple-truss spans ranges from 200 to 300 ft., according to the width of the structure.

From these various diagrams and the computations involved in their preparation, some interesting and useful deductions have been drawn.

Introduction

During the last fifteen years, the building of large and costly bridges over the navigable waters of the United States has been gradually on the increase, until, to-day (1932), there are projected, under construction, and completed, numerous structures of this type scattered throughout the entire country. When the spans required are excessively long, suspension bridges are adopted; and when they can be made reasonably short, simple-truss spans are used; but, in the majority of cases, where the rivers crossed are wide, the cantilever type of bridge is found to be the most suitable. Such being the case, an economic investigation concerning this class of structure is certainly in order; hence, this paper, upon which the writer has been working (in a rather desultory manner, it must be confessed), during the five years, 1926–1930, inclusive.

In addition to the writer's studies of the long ago (to be described later) concerning the economics of railway cantilever bridges, investigations have subsequently been made on the subject by the late Edgar Marburg, M. Am. Soc. C. E., and Henry S. Jacoby, William H. Burr, and D. B. Steinman, Members, Am. Soc. C. E., but their computations generally consisted of theoretical analyses based upon approximate assumptions, in order to simplify the mathematics involved. These assumptions do not represent with sufficient accuracy the actual governing conditions; hence the results of the various researches are not in accord. They need a check by some practical method; and it has been the object of this research to furnish such a check. The principal aim of these investigators was to determine the economic length of the suspended span in the usual type of three-span cantilever bridges; but, as indicated hereinafter, there are a dozen or more variables involved in the solution of the problem, and these, with their varying importance, must unavoidably produce divergent results.

History

About thirty-five years ago, while preparing the manuscript of "De Pontibus," the writer had his office force compute, for Chapter V thereof, a number of complete designs and estimates of quantities of materials, in order

to determine the economic functions and weights of metal for single-track railway, pin-connected, carbon-steel, cantilever bridges. The principal results of the investigation, as indicated in that little treatise, were as follows:

(*A*) The economic length of the suspended span, in an ordinary three-span structure, is three-eighths of the length of the main span, measuring from center to center of piers.

(*B*) Given the total distance between centers of anchorages in such a layout and *carte blanche* as to the location of the two main piers, the economic length of each anchor arm is two-tenths of the total distance between centers of anchorages.

(*C*) When given a fixed distance between centers of main piers, to determine the economic length for the anchor arms, it was found that, for æsthetic reasons, the length of each of the said arms should not be less than 20% of that of the main opening, or, say, 15% of the total distance between centers of anchorages, greater lengths than this minimum being preferable for appearance.

(*D*) The economic truss depth or tower height over main piers is about 15% of the length of the main span.

Present Study

The *vera causa* of this paper was a statement made about five years ago by the writer's partner, Shortridge Hardesty, M. Am. Soc. C. E., to the effect that the economic functions of modern, highway cantilever bridges with their comparatively great dead loads, small live loads, and wide roadways, are likely to differ materially from those of old-time, single-track railway cantilever bridges with their comparatively great live loads, small dead loads, and narrow roadways. In his various designs for long-span, highway cantilever structures, Mr. Hardesty had not been governed at all closely by the "De Pontibus" findings; hence, the writer's decision to make a new set of economic investigations for highway cantilever bridges instead of railway cantilever bridges and for modern, instead of old-time, conditions.

The principal economic functions that required determination were the following:

(*a*) Proportionate length of the suspended span, for an ordinary three-span layout, in terms of the main-span length, when the anchor arms are short.

(*b*) Proportionate length of the suspended span, for an ordinary three-span layout, in terms of the main-span length, when the anchor arms are long.

(*c*) The economic depth of truss over main pier in terms of the main-span length.

(*d*) For the ordinary, three-span, cantilever layout, given the positions of the two main piers, what is the economic length for the anchor arms? In comparing total costs of a group of layouts, the "out-to-out" lengths of all contrasted structures must be made equal by the addition of steel trestle approaches.

(*e*) For the ordinary, three-span, cantilever layout, given the locations of the anchor piers and *carte blanche* as to the two main pier locations, what is the economic proportionate length for the anchor arms?

(*f*) For a layout consisting of a suspended span, a cantilever arm, an anchor span, a like cantilever arm, and a like suspended span, given the positions of the end piers and *carte blanche* as to the two main-pier locations, what is the economic proportionate length for the anchor span?

Basic Data

A search of the writer's office records provided the necessary data from seven complete studies with estimates of quantities of materials and costs, namely, two Arthur Kill Bridges for the Port of New York Authority, each having a 42-ft. clear roadway and two 5-ft. clear sidewalks; one Mississippi River Bridge, and one proposed Ohio River Bridge, at Cairo, Ill., each having a 20-ft. clear roadway and no sidewalks; two Cooper River bridges, at Charleston, S. C., each having a 20-ft. clear roadway and no sidewalks; and a proposed Mississippi River Bridge at New Orleans, La., having a 40-ft. clear roadway and two 6-ft. sidewalks.

The main-span, side-span, and suspended-span lengths of these structures are as shown in Table 1.

TABLE 1.—Dimensions of Cantilever Bridges

Structures.	Main span, in feet.	Side spans, in feet.	Suspended span, in feet.
Perth Amboy (N. J.) Bridge over the Arthur Kill	750	375	301
Elizabeth (N. J.) Bridge over the Arthur Kill	672	240	336
Cairo (Ill.) Bridge (as first designed) over the Mississippi River	675	450	245
Cairo (Ill.) Bridge, as later designed	700	450	254.5
Proposed Cairo Bridge over the Ohio River	730	444	265
Charleston (S. C.) Bridge over the Cooper River	1 050	450	437.5
Charleston (S. C.) Bridge over the Cooper River, Town Creek Span	640	256	320
Proposed New Orleans (La.) Bridge over the Mississippi River	1 754*	493	714

* Since increased by the War Department to 1 760 ft.

Besides these data, there were available computations of quantities of materials made for the Perth Amboy Bridge of the Port of New York Authority, with a main opening of 800 ft., and estimates for some other spans of various lengths. It is fortunate that two of the recorded structures have main spans of almost exactly the same length, and that one of these has long anchor arms and the other short ones, because the writer has come to recognize that the economic length of the suspended span may be affected materially by the lengths of the anchor arms. On the other hand, however, it is unfortunate for this investigation that the width of roadway in three of the bridges should be about 40 ft., plus two sidewalks, and in the other five only 20 ft., without sidewalks, because there is more theoretically excessive material in narrow bridges than in wide ones; and, in addition, the wind loads add to the chord sections in a narrow structure, whereas usually they do not in a wide one.

Effect of Substructure

In determining the comparative economics of different layouts for any cantilever bridge, the variation in dimensions of both the main piers and

the anchor piers will affect somewhat the proportionate economic length of the suspended span; but these effects usually cannot readily be forecast or properly evaluated. If the piers rest on bed-rock, the changes in loading on pier tops, even for large variations of length in the suspended span, may cause no increase or decrease of importance in the total cost of substructure, because, in highway bridges, the sizes of piers are often governed by a sense of fitness rather than by the superimposed loads; but where the piers rest on piles or comparatively soft material, their cost will generally be an almost direct function of the superimposed loading.

Short-Cuts in Making Calculations

If, at the outset of this investigation, it had been necessary to prepare a complete detailed estimate of cost for the superstructure of each layout used in locating points on the economic curves, the labor and expense involved would have been too discouraging. Therefore, some "short-cuts" were adopted, so as to secure, with sufficient accuracy for general purposes, the results needed for plotting at least four points on each of the curves. To this end the writer made the following assumptions that he deemed close enough to theoretical correctness for the object in view:

First.—Variations in metal weights from bending and from shear, when passing from known values in one layout to the corresponding desired values in another and somewhat similar layout, are almost in the same proportion; hence no great error would be involved in assuming that bending moments alone will constitute the working criteria for economic investigation.

Second.—After all the bending moments for any layout of spans are plotted to scale, and the greatest direct and the greatest reverse moments for dead load, live load, and impact are computed, then, on the principle of adding one-half of the smaller of two moments of opposite sign to the larger one, the sum of the three moment areas for the layout, when compared with the corresponding sum for another similar but slightly different layout, will give a ratio nearly identical with the ratio of the weights of truss metal in the two layouts.

On this basis the economic length of the suspended span, as far as the superstructure is concerned, can be determined by plotting the moment areas; and from these, all the needed weights of truss metal can be found rather expeditiously, and with sufficient accuracy for the purpose contemplated, by using ratios of average moments.

Third.—While, in computing the dead-load moments over the main pier, due cognizance is taken of the correct center of gravity of the dead load in the cantilever arm, in order to avoid complicated plotting of moments, these are assumed to vary according to the ordinates of a triangle, as are also the moments for the live load on the cantilever arm. This is equivalent to assuming that the entire moment over the main pier is due to an undetermined loading placed at the end of the cantilever arm, and that the latter is stressed accordingly. While the error involved by this assumption may

not be small, it applies with about equal seriousness to not too different layouts; hence, the differential of the error may be considered negligible in computing the moment areas.

Fourth.—Similarly, in computing the dead-load moments on the anchor arm, cognizance is taken of the center of gravity of the dead load; nevertheless, the amount of the maximum moment is calculated somewhat as if the dead load were uniformly distributed. The moments are plotted by means of two half parabolas of the same height, but of different lengths, the axis of division passing through the center of gravity. As in the last case, the effect of the incorrectness of this assumption upon the comparison of total areas is not large.

Economic Lengths of Suspended Spans

In order to determine the economic lengths of suspended spans, two sets of computations were made, one for short anchor arms, and one for long anchor arms, the moment areas were plotted, and the economic span lengths

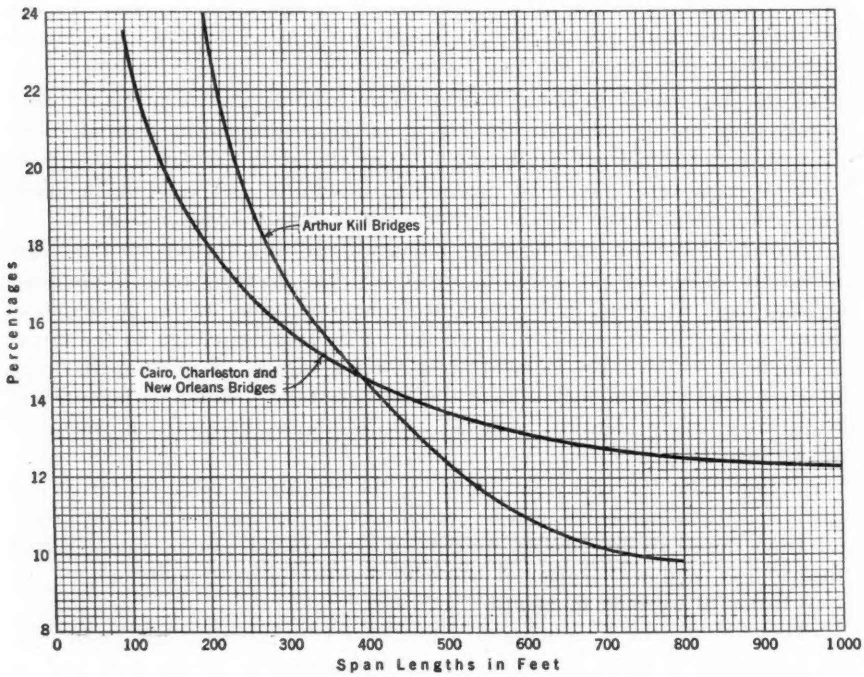

Fig. 1.—Total Live Loads per Linear Foot of Span for the Elizabeth and Perth Amboy Bridges, in New Jersey.

were determined, all on the basis of "guessed" weights of metal for trusses in the cantilever arms and the anchor arms of the uncomputed layouts; but accurate data were available from office records for the corresponding weights in the suspended spans.

A check of the assumed weights of metal showed them to be fairly accurate, but a few of the moment plots had to be slightly modified to suit the final, corrected dead loads. In making these new assumptions, the average moments were computed from the preliminary moment diagrams that were made for temporary use; and, in passing from a layout of known weights

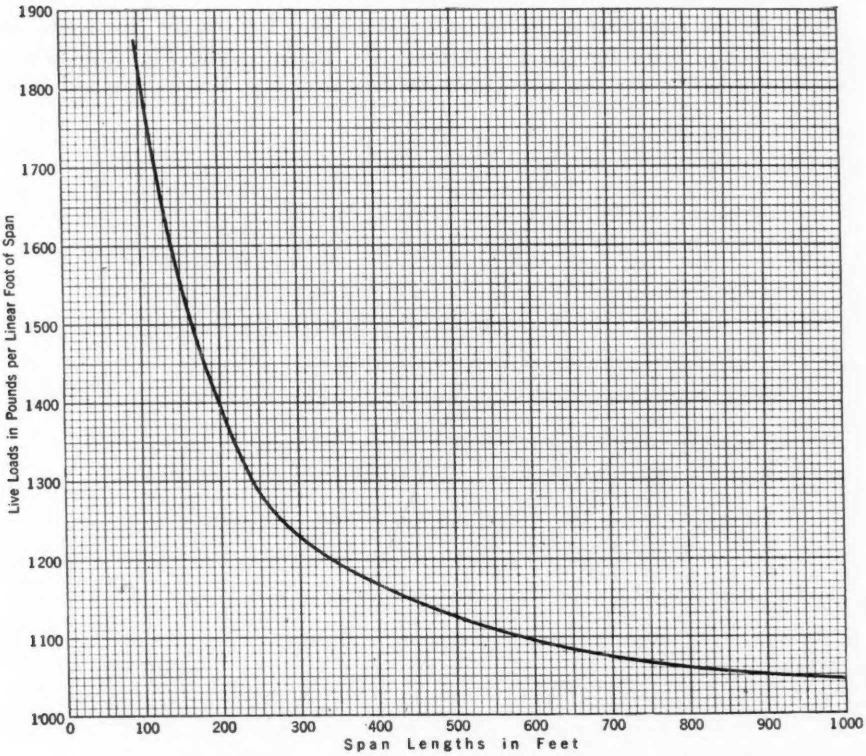

FIG. 2.—TOTAL LIVE LOAD PER LINEAR FOOT OF SPAN FOR THE MISSISSIPPI BRIDGE AT CAIRO, ILL.

of truss metal to a closely similar layout, the corresponding new weights were found by a modification of the writer's long established equation,

$$W' = \frac{W}{5} (1 + 4r) \qquad (1)$$

or,

$$W' = \frac{W}{7.5} (1 + 6.5r) \qquad (2)$$

in which, W' is the new average weight of truss metal per linear foot of span, W, the corresponding old weight, and r, the ratio of the average moments for their spans.

This correction made little difference in the economic curves first determined; for, as before indicated, the variations from the assumed deal loads were comparatively small.

Data from Computed Spans

The data for several of the computed structures previously mentioned are shown in the following diagrams and tables: Fig. 1 gives the live loads per linear foot of span for the two Arthur Kill Bridges; and Fig. 2 furnishes that information for the two Cairo Bridges over the Mississippi and Ohio

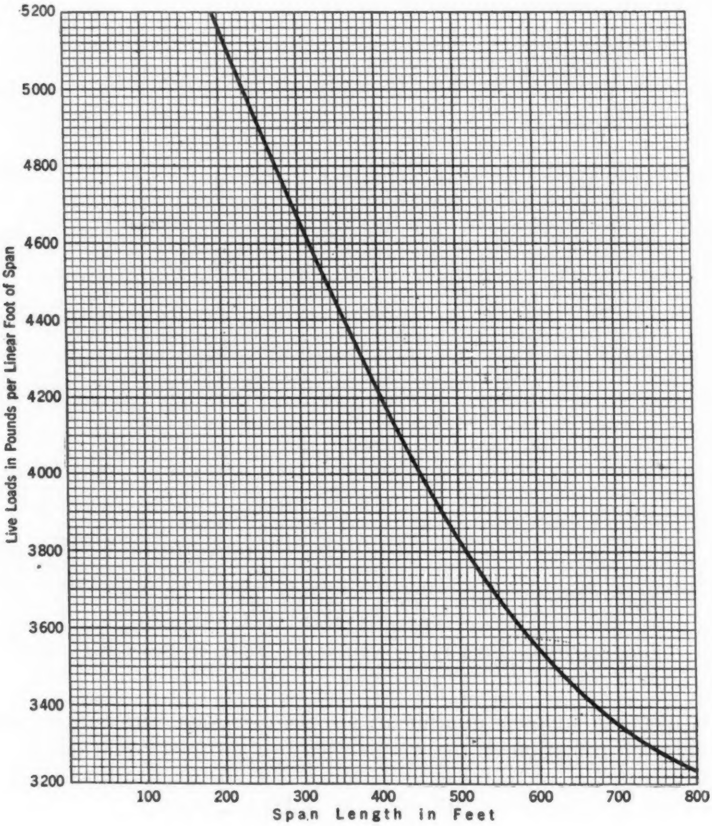

FIG. 3.—IMPACT PERCENTAGES

Rivers, and for two of the spans of the Cooper River Bridges, at Charleston. Fig. 3 shows the impact percentages for all these structures.

Weights of metal per linear foot of span, and the dead loads that were used as a basis in making the computations for this paper, are recorded in Table 2.

New Computations

Using the data from Table 2, the writer computed from the Elizabeth Bridge statistics numerous curves of bending moments similar to Fig. 4, for various ratios, *r*, of suspended span to main span. The total moment-area

TABLE 2.—Distribution of Weights of Materials, in Pounds per Linear Foot of Span, in Highway Cantilever Bridges

Name of structure and its main-span length	Portion of structure	Anchor Arm		Cantilever Arm		Suspended Span	
		Metal	Flooring	Metal	Flooring	Metal	Flooring
Elizabeth Bridge, 672 ft.	Deck and conduits	......	8 500	......	8 500	......	8 500
	Hand-rails, etc.	120	......	120	......	120	......
	Floor system	2 000	......	1 950	......	1 950	......
	Laterals	810	......	900	......	650	......
	Trusses	4 540	7 470	5 560	8 530	3 050	5 770
	Shoes and anchors	560	......	280	......	......	......
	Total metal	8 030	......	8 810	......	5 770	......
	Dead load on trusses	......	15 970	......	17 030	......	14 270
Perth Amboy, Bridge, 750 ft.	Deck and conduits	......	8 500	......	8 500	......	8 500
	Hand-rails, etc.	120	......	120	......	120	......
	Floor system	1 860	......	1 860	......	1 830	......
	Laterals	830	......	970	......	720	......
	Trusses	4 880	7 690	6 300	9 250	2 600	5 270
	Shoes and anchors	320	......	280	......	......	......
	Total metal	8 010	......	9 530	......	5 270	......
	Dead load on trusses	......	16 190	......	17 750	......	13 770
Cairo Bridge, 700 ft.	Deck	......	2 360	......	2 360	......	2 360
	Hand-rails, etc.	90	......	90	......	90	......
	Floor system	640	......	640	......	640	......
	Laterals	320	......	380	......	270	......
	Trusses	2 000	3 050	2 400	3 510	1 210	2 210
	Shoes and anchors	100	......	100	......	......	......
	Total metal	3 150	......	3 610	......	2 210	......
	Dead load on trusses	......	5 410	......	5 870	......	4 570
Charleston Bridge, 1 050 ft.	Deck	......	2 100	......	2 100	......	2 100
	Hand-rails, etc.	100	......	100	......	100	......
	Floor system	850	......	850	......	850	......
	Laterals	550	......	660	......	380	......
	Trusses	3 120	4 620	3 610	5 220	1 770	3 100
	Shoes and anchors	120	......	120	......	......	......
	Total metal	4 740	......	5 340	......	3 100	......
	Dead load on trusses	......	6 720	......	7 320	......	5 200
Charleston Bridge, 640 ft.	Deck	......	2 100	......	2 100	......	2 100
	Hand-rails, etc.	80	......	80	......	80	......
	Floor system	640	......	640	......	640	......
	Laterals	330	......	420	......	250	......
	Trusses	1 740	2 790	2 150	3 290	1 280	2 250
	Shoes and anchors	100	......	100	......	......	......
	Total metal	2 890	......	3 390	......	2 250	......
	Dead load on trusses	......	4 890	......	5 390	......	4 350

for the anchor arm, the cantilever arm, and one-half the suspended span was marked at the bottom of each sheet. These total moment areas were then plotted in Fig. 5. From that diagram it is seen that the economic length of the suspended span, as far as superstructure metal alone is con-

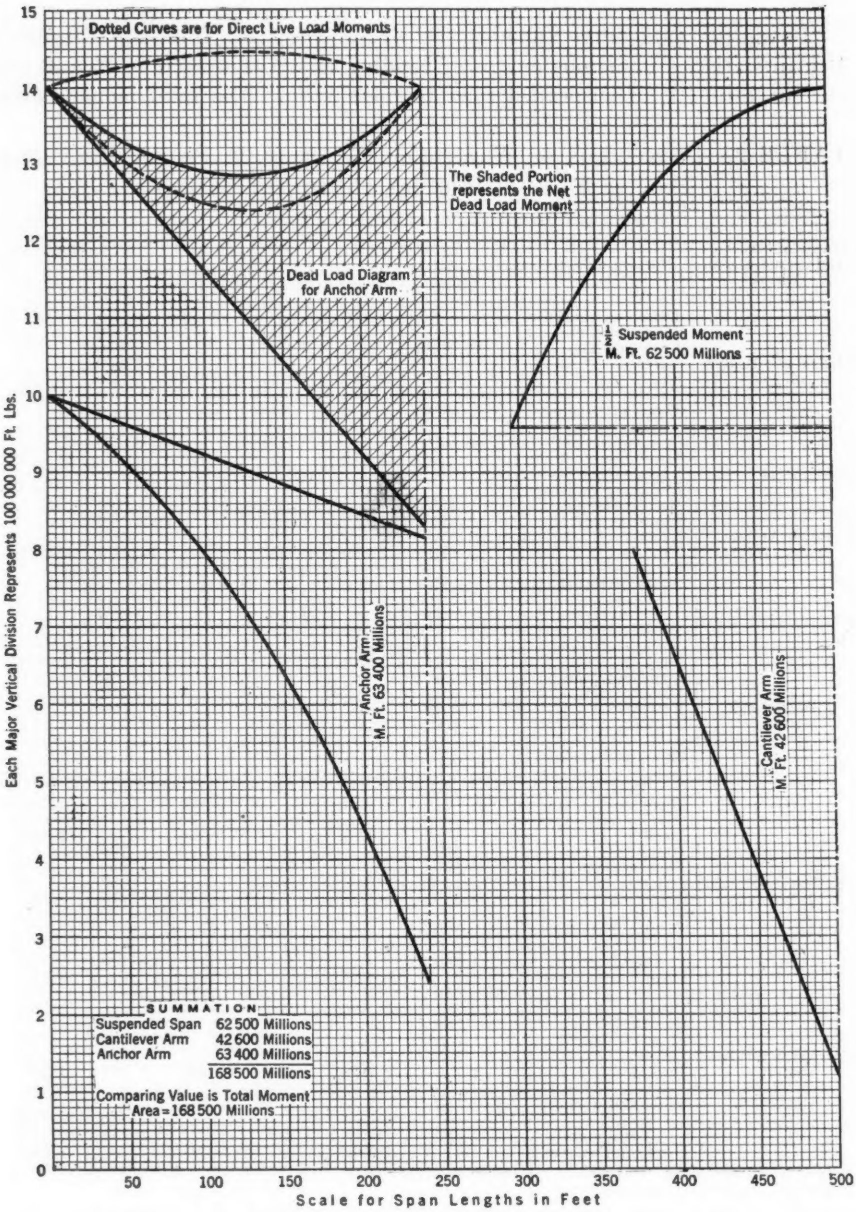

FIG. 4.—MOMENT AREAS FOR ELIZABETH, N. J., BRIDGE (r=0.625).

cerned, is theoretically about 60% of the main-span length, measured from center to center of piers.[2]

Similar curves of bending moments were computed from the Cairo Bridge statistics (for example, see Fig. 6), and, as before, at the bottom of each sheet the value of the comparing moment-area was written; then, these moment-areas were plotted in Fig. 7. From that diagram it is seen that

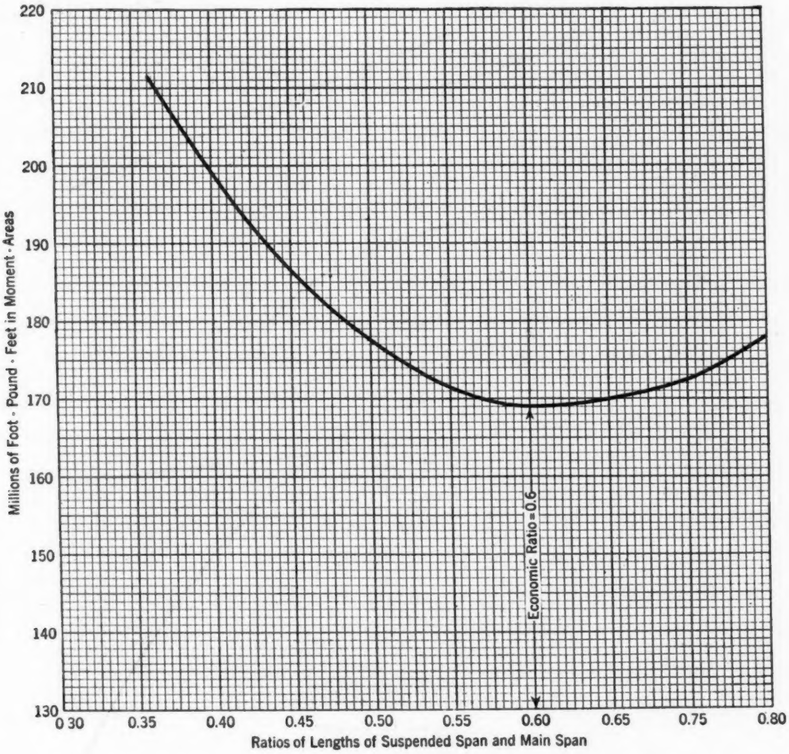

FIG. 5.—ELIZABETH, N. J., BRIDGE: MOMENT AREAS FOR VARIOUS RATIOS OF LENGTHS OF SUSPENDED SPAN TO MAIN SPAN.

the economic length of the suspended span, as far as superstructure metal alone is concerned, is theoretically about 52% of the main-span length, measured from center to center of piers.

The variation in the economic percentages as shown in Figs. 5 and 7 is due mainly to the great difference in the lengths of the anchor arms in the two cases; but it may be influenced slightly by the difference in the widths of deck. The result confirms the writer's anticipated variation.

[2] As this set of curves has already served its purpose and can be of no further use to the profession, except to afford an opportunity for checking the writer's accuracy and good faith, only a single specimen thereof is herein published; but a complete set of blue prints of all the diagrams of the paper will be placed on file in the Engineering Societies Library, 33 West 39th Street, New York, N. Y.

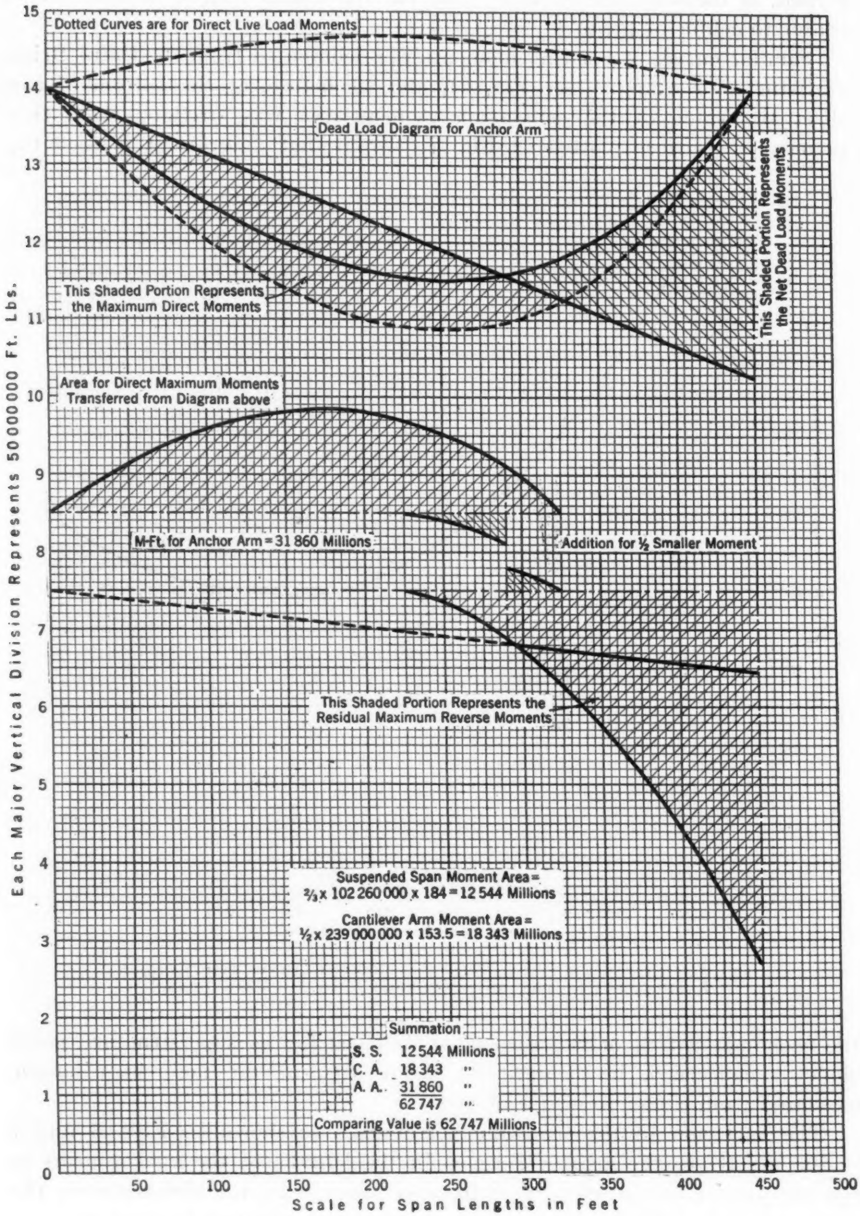

FIG. 6.—MOMENT AREAS FOR CAIRO, ILL., BRIDGE.

It will be noticed that the economic curves of Figs. 5 and 7 are very flat at the bottom, so that for a variation of several points on each side of the minimum there is almost no difference in the moment areas, and, consequently, also in the weights of metal; hence, it will be a wise precaution to reduce somewhat the indicated minima, making them, respectively, 55 and 48 per cent. The reason for so doing is that certain influences, which were not considered in the investigation, tend to reduce materially the economic length of the suspended span; for instance, the erection stresses in its trusses,

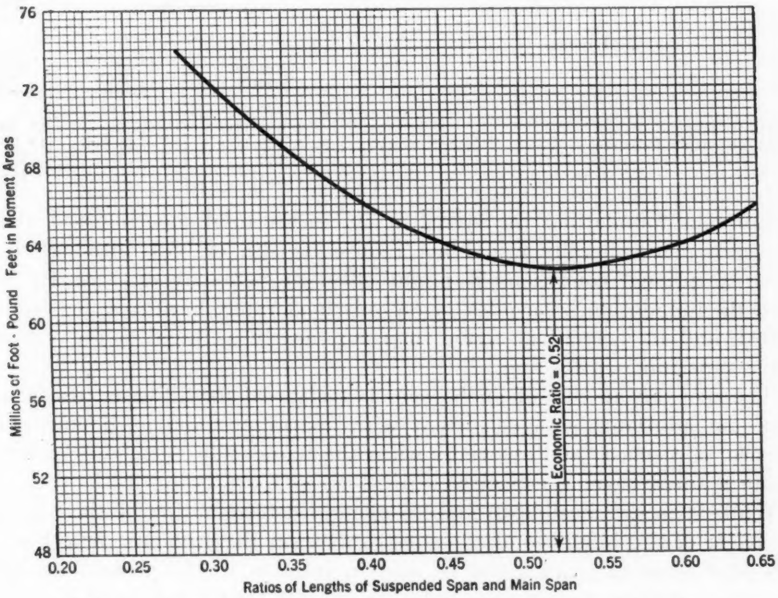

FIG. 7.—CAIRO, ILL., BRIDGE: MOMENT AREAS FOR VARIOUS RATIOS OF LENGTHS OF SUSPENDED SPAN TO MAIN SPAN.

when these are cantilevered, are likely to augment some of the sectional areas of the members, as are also the wind loads (especially in narrow structures). Wind loads and variations in weights of erection toggles have been ignored in this investigation, so as to save labor in making computations.

EFFECT OF SUBSTRUCTURE VARIATION

The question as to how much the variation in cost of the substructure affects the economic length of the suspended span is an unsatisfactory one to solve, for a great deal will depend on the character of the foundations and on the massiveness or slenderness of the pier shafts. With piers on solid rock and shafts made as small as is consistent with both appearance and good engineering practice, the effect would generally be nil; but if the pier tops have to be changed to suit the superimposed loads, a slight effect would be found, even with pier bases on solid rock. When, however, the bases are supported on piles, or on soil that is only moderately hard, the effect undoubtedly must be considered.

It is recognized, of course, that lengthening the suspended span reduces the superimposed loads on the main piers, decreases the uplift on the anchorages when the anchor arms are short, and increases the direct load thereon when the anchor arms are long.

Data for the Elizabeth Bridge from the calculations made for this paper are given in Table 3.

TABLE 3.—SUPERIMPOSED LOADS, IN POUNDS, ON PIER TOPS.

Layouts.	Anchor Piers.		Main Piers
	Greatest uplift	Greatest direct loading	Greatest direct loading
For 336-ft. suspended span	1 684 000	None	13 100 000
For 420-ft. suspended span	1 159 000	146 000	12 200 000
Ratios	$r = \frac{1159}{1684} = 0.69$	....	$r = \frac{122}{131} = 0.93$

For rock foundations the saving of 7% in the total superimposed load on the main pier generally would not reduce its size or cost, although under some exceptional conditions it might effect an economy of 3 or 4%; but, when the foundation is on piles, or on only moderately hard soil, the economy is real, and its percentage value may run nearly as high as the variation in the superimposed loading.

The economy in cost of anchorages is truly real for all cases, except only when the minimum size, consistent with appearance and good engineering practice, is reached; and especially when the foundation is on piles or on soil having a small bearing capacity.

While it is not feasible to make a general determination of the quantitative effect of the influence of the substructure on the economic length of the suspended span, this much can be said in relation to the present investigation—that, if there is any such influence, it will tend to augment the length herein determined.

Effect of Method of Erection

It should be remembered that the preceding superstructure investigation is based on the assumption of "floating in" the suspended spans, thus avoiding the use of any extra truss metal to resist erection stresses. The probable

TABLE 4.—ECONOMIC PERCENTAGES FOR SUSPENDED-SPAN LENGTHS COMPARED WITH MAIN-SPAN LENGTHS

Comparative length of anchor arm	For cantilever erection	For flotation erection
Long	43	48
Short	50	55

effect on the economic length from erection by cantilevering would be to reduce the ratio by about five points, thus making it 43% for structures with long anchor arms and 50% for those with short anchor arms. The final results of the preceding dissertation are given in Table 4.

In the case of very long suspended spans floated into place, it may be economical to shorten them considerably below the theoretical economic length, because of the great expense for barges and hoisting machinery. This is likely to make a difference of as much as five points, or even more under extreme conditions.

Again, in the case of the erection of the suspended span by cantilevering, when eye-bars are used in the bottom chords, it will be necessary to stiffen temporarily these purely tension members; hence a reduction of the percentages given in Table 4 would be logical, say, about five points.

Verification by Section Diagrams

Fearing that the conclusions from the preceding calculations (because of the use of the moment-area method) may not prove acceptable to the profession, the writer decided to make a test that is reliable beyond any doubt, namely, the comparison, for the trusses of two layouts having different lengths of suspended span, of $\Sigma\ l\ a$ for the anchor arm, the cantilever arm, and one-half the suspended span, in which l is the axial length, in feet, of any truss member and a is the gross area of its cross-section, in square inches, Σ, of course, being the summation sign, in order to observe how the comparison agrees with that obtained by the moment-area method.

TABLE 5.—Comparison of Economy of Polygonal Top Chords and Parallel Chords.

Span.	Suspended Span, Ratio=0.5		Suspended Span, Ratio=0.625	
	Parallel chords.	Polygonal top chords.	Parallel chords	Polygonal top chords
Anchor arm	107 418	107 418	94 192	94 192
Cantilever arm	113 796	113 796	88 733	88 733
One-half suspended span	63 130(a)	62 374(b)	95 039(c)	90 549(d)
Totals	284 344(A)	283 588(B)	277 963(C)	273 474(D)

Applying this to two of the layouts for the Elizabeth Bridge, in both of which the chords of the suspended span are parallel, there resulted a saving of $2\frac{1}{4}\%$ by the longer suspended span. This result is not absolutely conclusive, however, because for true economy the top chords of the latter should be polygonal; consequently, the writer had one of his assistant engineers recompute the two layouts, using polygonal top chords and adopting the most economic truss depths for all spans in both cases, as closely as the experience of the office force and himself could determine. The results of these various calculations, in pounds of metal, were as shown in Table 5.

The savings indicated are, as follows:

$$(1)\ldots\ldots\ \frac{a-b}{a} = 1.2 \text{ per cent.}$$

$$(2)\ldots\ldots\ \frac{c-d}{c} = 4.7 \text{ per cent.}$$

$$(3) \ldots\ldots \frac{A - C}{A} = 2.2 \text{ per cent.}$$

$$(4) \ldots\ldots \frac{B - D}{B} = 3.6 \text{ per cent.}$$

A comparison of Item (1) with Item (2) indicates how rapidly the economy of polygonal top chords over parallel chords in simple-truss spans increases with the span length.

Referring to Fig. 5, the ratio of moment areas for $r = 0.600$ and $r = 0.5$, is $\frac{169}{180} = 0.952$, showing a saving of 4.8% instead of the 3.6% found in Item (4). This indicates that, although the theoretical economic span length determined by the moment-area method is closely correct, the actual variations in economics for longer and shorter spans are not quite as rapid as those found by that method of computation. In any event, it has been proved that the moment-area method of comparison is accurate enough for practical purposes; consequently, it will be used for the solution of the remaining economic problems of this research.

Tabulation of Results to Present Stage

Before passing to such additional problems, however, it will be well to compare the ratio results already found with those of other investigators, as follows:

Henry S. Jacoby......	....	= 0.68
Edgar Marburg	0.6 to 0.7, average...........	= 0.65
William H. Burr......	0.5 to 0.55, average...........	= 0.53
D. B. Steinman.......	....	= 0.60
J. A. L. Waddell......	0.43, 0.48, 0.50, 0.55, average...	= 0.49

The average of the averages for the four other investigators is 0.615, while that of the writer is 0.49. The general practice of the writer's firm is to use ratios somewhat smaller than those indicated in Table 4, one reason being the addiction of his office force to the use of the Warren or triangular truss rather than the Pratt or Petit truss. An investigation of this question was made as a check, by M. N. Quade, Jun. Am. Soc. C. E., using the 1 050-ft. main span of the Cooper River Bridge as a basis. Mr. Quade computed six curves, each having three points, using not only the 20-ft. clear roadway of the structure, but also an assumed one of 40 ft., taking carefully into account the effects of wind loads, erection stresses, and minimum sections. He found the economic lengths of the suspended span to vary from 0.44 to 0.46 of the length of the main span, with an average of 0.45; but his curves were very flat between the 0.42 and the 0.05 ratios, the average excess weight of metal for the latter minimum being only 1.2 per cent. This shows that, in adopting 0.5 as a working value for the subsequent economic investigations, no error of importance was made. The Warren truss does not accommodate itself readily to changes in span length. For instance, in the Cairo

Bridge over the Mississippi River, as finally designed, the ratio is 0.36, there being eight panels of 32 ft. each; but if the same style of truss and the same panel length had been used for a longer suspended span, the governing conditions would have forced the adoption of twelve panels and a ratio of 0.54, which did not produce a pleasing layout—at least, to the eye of the designer.

From what precedes, one might be inclined to draw the deduction that the economic length of the suspended span, in respect to total weight of metal in a structure, is a rather uncertain quantity; and in so doing he would be perfectly right, for it is more or less dependent upon the following conditions: Width of roadway; method of erection; type of truss; width of main opening; æsthetics of layout; relative importance of wind loading; proportion of minimum sections involving theoretically excessive metal; proportion of heat-treated metal used, and where; proportionate utilization of alloy steel or steels; proportionate length of anchor arms; anticipated difficulties in erection; effect of substructure costs; use or non-use of eye-bars; and idiosyncracies of the computer.

In view of all these variables, it certainly is difficult to determine for any particular crossing the exact length of suspended span that will involve the absolutely smallest weight of metal for the entire structure; but it will be helpful to remember that it will generally be from 45 to 50% of the length of the main span.

Economic Considerations Other Than Metal Weights

It must not be forgotten that there are important factors other than total weight of metal that influence the economics of a cantilever layout, especially those relative to cost of erection, which usually have a tendency to shorten the suspended span. There is also the variation in costs of substructure, which generally has the opposite effect. Each case as it arises will have to be considered as a special economic problem, the data of this paper being used as a guide rather than a hard-and-fast rule in determining the layout of spans.

Economic Depth of Truss Over Main Piers

As previously mentioned, in some special investigations for "De Pontibus," the economic truss depths over the main piers for railway cantilever spans was ascertained to be 15% of the main-span length. Dr. Yun Tieng Chang found[3] exactly the same result for modern, highway cantilever bridges as is shown in Table 6.

It will be seen from Table 6 that a variation of unit percentage on either side of the minimum makes little difference in the total weight of metal; and, as a short column is easier to erect than a long one, it might be well to adopt 14% of the main opening as the economic truss depth. The writer's office force has found the best depth to vary between 12 and 15% with an average of 13.5 per cent.

[3] In his post-graduate thesis for his Doctor's degree, which was prepared in the writer's office.

The longer the suspended span the smaller will be the maximum moment over the main pier, and, consequently, the less will be the best depth there for the trusses. Furthermore, the wider the roadway and the heavier the bridge, the greater, most likely, will be the best truss depth. Again, the Petit truss generally calls for a slightly greater height than the Warren or triangular truss, appearance often being the governing factor.

TABLE 6.—WEIGHT OF BRIDGE WITH DIFFERENT TOWER HEIGHTS

Ratio: Tower height / Main-span length	WEIGHT, IN KIPS			
	Anchor arm	Cantilever arm	Suspended span	Total
0.13	1 757	610	710	3 077
0.14	1 500	580	710	2 790
0.15	1 425	570	710	2 705
0.16	1 495	565	710	2 770
0.17	1 773	610	710	3 093

ECONOMIC LENGTHS FOR ANCHOR ARMS WITH FIXED LENGTH OF MAIN SPAN

No development in the thirty-three years since 1898 has given the writer any reason for changing his conclusion concerning the best length for the anchor arms of any ordinary, three-span, cantilever bridge, when the main span length is fixed, namely, that for economy in total metal weight of structure it should be made as short as the governing conditions of layout will permit; but that for appearance it should not be less than one-fifth of the main opening or, preferably, a little longer.

ECONOMIC LENGTHS FOR ANCHOR ARMS WITH FIXED DISTANCE BETWEEN ANCHOR PIERS

As before stated, the "De Pontibus" research determined the economic length for anchor arms, when the distance between the anchor piers of an ordinary, three-span, cantilever railway bridge is fixed, to be one-fifth of the total length of the structure, or one-third of the main-span length. Dr. Chang found a somewhat different result, as shown in Table 7.

TABLE 7.—WEIGHTS OF METAL FOR DIFFERENT RATIOS OF ANCHOR ARM TO MAIN SPAN

(Weight, in kips.)

Ratio: Anchor arm / Main span	Total.	Minimum	Difference	Weight increase, in percentage
0.3	4 345	2 710	1 635	60.3
0.4	3 081	2 710	371	13.7
0.5	2 726	2 710	16	0.59
0.6	2 758	2 710	48	1.77
0.7	2 912	2 710	202	7.45
0.8	3 133	2 710	423	15.6
0.9	3 393	2 710	638	25.2

This shows that, for economy, the anchor arm should be about one-quarter the total length of the structure, or one-half the main-span length. This variation from the writer's old finding is undoubtedly due to the fundamental difference between the two types of structure investigated. The appearance of the layout would certainly be more pleasing with the 0.25 ratio than with the 0.2 ratio originally found for railway bridges.

The problem, however, is one that is not likely to arise often in actual practice, as there are usually conditions of ground or stream that determine the location of the main piers; nevertheless, it is a possibility worthy of consideration, for the ruling limitations might be such as to give the designer *carte blanche* in respect to main-pier locations.

Structure Having a Central Anchor Span, Two Cantilever Arms, and Two Suspended Spans

As generally the "most economic possible" type of cantilever layout consists of a structure having a central anchor span, two cantilever arms, and two suspended spans, it is likely that there will occasionally arise in a bridge engineer's practice the problem of determining the economic layout for such a bridge, when the positions of the outer piers are fixed, and when entire liberty is given as to the location of the two intermediate piers. In such a case the length of the suspended spans should be determined from the already ascertained percentage of the length of an assumed main span in an ordinary three-span cantilever having a length equal to $S + 2C$, in which, S is the length of the side suspended span and C is the length of the cantilever arm to which it is hung. The total length of the structure, L, is $A + 2C + 2S$, in which, A is the length of the anchor span, all as shown in Fig. 8, in which (for convenience in computation) S is taken equal to $2C$, which is nearly, if not quite, the most economic proportion.

The question for solution is: "Given the length, L, what is the economic length for A"? Before attempting to solve it, there is a preliminary investi-

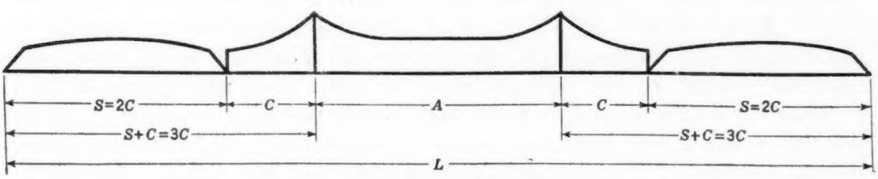

Fig. 8.—Type C Cantilever Bridge Layout.

gation that can be made advantageously, that is, "having a layout of spans resembling that illustrated in Fig. 8, and with a fixed length for the side opening ($S + C = 3C$) and certain standard live loadings with all the corresponding truss-metal weights per linear foot of span shown for S and C, what are the weights per linear foot for adjacent A spans of various lengths"? At present, these proportionate weights are unknown to the profession and are sometimes needed.

For the investigation, the suspended span and cantilever arms of the Elizabeth Bridge can be used, for which the weights of metal have been already recorded, and in which, $S = 2C = 336$ ft. and $C = 168$ ft. Assume that $A = 300$ ft., $A = 400$ ft., $A = 500$ ft., $A = 600$ ft., and $A = 700$ ft., and find by the moment-area method the weights of metal in the trusses per linear foot of span, the weight per foot of floor metal being constant, and that of the lateral system increasing directly with the span length.

Anchor Span: 300-Foot

Using the weights of metal per linear foot of span for the Elizabeth Bridge, as recorded in Table 2, and the live loads and the impact percentages shown in Figs. 1 and 3; assuming a trial truss weight of 12 400 lb. for the anchor span; computing the average moments for both the anchor span and the suspended span, and finding their ratio, the truss weight for the anchor span was ascertained to be 12 250 lb. per lin. ft., which is close enough to the trial weight. It is almost needless to state that it took more than one assumption to obtain a coincidence even as close as that.

Anchor Spans: 400, 500, 600, 650, and 700-Foot

Similar calculations resulted in finding the following weights of truss metal per linear foot of span for longer anchor spans:

Length of anchor span, in feet	Weight of metal in trusses, in pounds per linear foot	Length of anchor span, in feet	Weight of metal in trusses, in pounds per linear foot
300	12 250	600	7 850
400	10 300	650	11 115
500	8 320	700	13 970

The standard yardstick for cantilever bridges is the main-span length, or the distance between centers of inner piers, in an ordinary three-span structure, which consists of two anchor arms, two cantilever arms, and a suspended span. Hence, it will be well to refer all truss weights to the average of those of the suspended span and the two cantilever arms, it being understood that the weights of the columns over the main piers are equally divided between the cantilever and anchor arms. It will facilitate computations, both for this research and for future bridge computers, if a satisfactory relation can be established between main-span lengths and the average weights of metal per linear foot for the trusses of the main spans. From data obtained in designing the Cairo and the Charleston Bridges (all spans being of the same general type and loading, and proportioned by the same specifications), the writer has evolved the equation:

$$w' = w \; \frac{(r + r^2)}{2} \qquad (3)$$

in which, w and w' are the average weights of metal per linear foot, for the trusses of the two main openings considered, namely, l and l', in feet, and

$r = \frac{l'}{l}$. Equation (3) is fairly accurate, provided the lengths, l and l', do not differ too greatly.

Again, for layouts like that of Fig. 13, when changing the length of the main span from l to l', the new weight of truss metal per linear foot in the suspended span, w'_s, is given by the equation,

$$w'_s = r\,w_s \qquad (4)$$

In the same type of layout when making the change, the new average weight of truss metal per linear foot in the cantilever arm, w'_c, is given by the equation,

$$w'_c = w_c \frac{(r + 3r^2)}{4} \qquad (5)$$

Returning to the layout in Fig. 8, if w_m is the average weight of metal per linear foot in the trusses for a completed main span, equal in length to

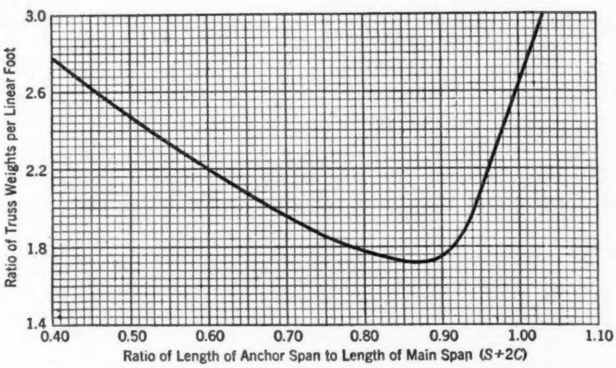

FIG. 9.—RATIOS OF WEIGHTS OF METAL PER LINEAR FOOT IN TRUSSES OF ANCHOR SPANS AND MAIN SPANS.

$S + 2C$ (or, in this particular case, to $4C$, or 672 ft.), a diagram can be made of ratios of weights of truss metal per linear foot in that main span and in anchor spans of various lengths. This was done and is recorded in Fig. 9.

The reader is now ready to proceed with the solution of the previously mentioned problem: "Given the total length, L, in the layout shown in Fig. 8, what is the economic length, A, of the anchor span"?

Still using the Elizabeth Bridge as a basis, $L = A + 6 \times 168$ ft., and assuming at first that $A = 400$ ft., then $L = 1\,408$ ft.; and the truss weights, in pounds per linear foot, will be as follows:

Suspended span	3 050
Cantilever arm	5 560
Main span ($S + 2C$)	4 300
Anchor span	9 460

The total weight of truss metal in the entire structure, therefore, will be,

$$W = 2 \times 336 \times 3\,050 + 2 \times 168 \times 5\,560 + 400 \times 9\,460 = 7\,702\,000 \text{ lb.}$$

Assume that $A = 450$ ft.; then, $C = \frac{1\,408 - 450}{6} = 159.7$, and $S = 319.4$; also, $r = \frac{638.8}{672} = 0.95$; $r^2 = 0.9025$; $\frac{r + r^2}{2} = 0.926$; and $\frac{r + 3\,r^2}{4} = 0.914$.

The truss weights per foot in the various spans are:

$$
\begin{aligned}
S &= 3\,050 \times 0.95 = 2\,898 \\
C &= 5\,560 \times 0.914 = 5\,082 \\
S + 2C &= 4\,300 \times 0.926 = 3\,982 \\
A &= 3\,982 \times 1.95 = 7\,765
\end{aligned}
$$

Hence,

$$w = 2 \times 319.4 \times 2\,898 + 2 \times 159.7 \times 5\,082 + 450 \times 7\,765 = 6\,969\,000 \text{ lb.}$$

Similar computations were made with anchor arms of 500, 520, 550, and 580 ft., for which the results were as follows:

Length of anchor span, in feet	Total weight of metal, in pounds, in trusses for entire bridge	Length of anchor span, in feet	Total weight of metal, in pounds, in trusses for entire bridge
400	7 702 000	520	6 100 000
450	6 969 000	550	6 761 000
500	6 182 000	580	8 456 000

In Fig. 10 the abscissas are the ratios of length of anchor arm to total length of structure, and the ordinates are the total weights of truss metal in

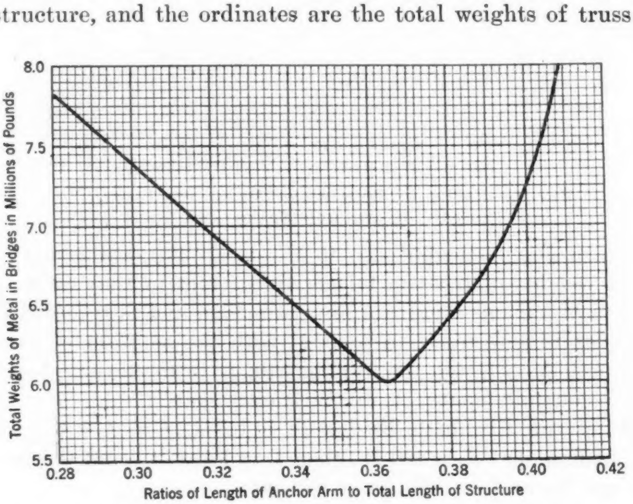

FIG. 10.—TOTAL WEIGHTS OF METAL IN STRUCTURE FOR TYPE C CANTILEVER BRIDGES, WITH VARIOUS LENGTHS OF ANCHOR SPAN

the bridge. The economic length of the anchor arm is well defined as about 36% of the total length of the structure; and the sharpness of the **U**-curve indicates that any material variation from this ratio would be decidedly uneconomic.

This last investigation has led up to and permitted the preparation of Fig. 11, in which are recorded the weights of metal per square foot of floor for modern highway cantilever bridges of Type C[4]. The layout is indicated in Fig. 8, and the metal is mainly silicon steel.

The *modus operandi* of computing these weights was, as follows: In Fig. 8, let $A = xC = 0.36\ L$. Then, $L = \dfrac{xC}{0.36}$; but $6C + xC = L = \dfrac{xC}{0.36}$. Therefore, $6 + x = \dfrac{x}{0.36}$, and $x = 3.37$. Hence, $L = 9.37\ C$.

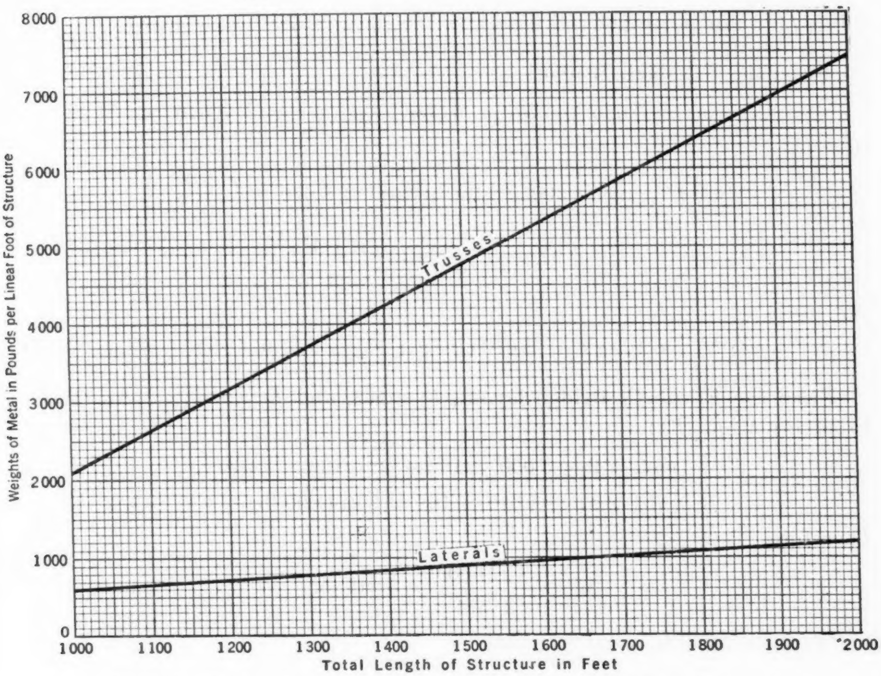

FIG. 11.—PRELIMINARY DIAGRAM OF WEIGHTS PER LINEAR FOOT FOR CANTILEVER HIGHWAY BRIDGES OF TYPE C

Still using the Elizabeth Bridge, for which, $C = \dfrac{672}{4} = 168$ ft., let the weights of metal per linear foot in trusses be, as follows: For the suspended span, w_s; for the cantilever arm, w_c; and for the main opening, $\dfrac{w_s + w_c}{2}$.

Using Fig. 9 for the ratio, R (on the abscissa line), $\dfrac{A}{4\,C} = \dfrac{3.37\,C}{4\,C} = 0.84$. For this, the ratio of truss weights per foot is 1.76; therefore.

$$W_a = 1.76\ \frac{(w_s + w_c)}{2} = 0.88\ (w_s + w_c).$$

[4] "Bridge Engineering," p. 1271.

Let w_l = the average truss weight per linear foot for the entire structure; then,

$$w_l\,L = w_s \times 2C + \frac{w_s + w_c}{2} \times 4C + \frac{w_s + w_c}{2} \times 1.76 \times 3.37\;C$$

or,

$$w_l\,L = 2\,w_s\,C + 2C\;(w_s + w_c) + 2.97\;(w_s + w_c)\;C \;\;\ldots\ldots\ldots(6)$$

Taking a new total length, L', with $\frac{L'}{L} = r$, Equation (6) can be written:

$$w'_l\,L' = 2rw_s\,C' + 2C'\;(w_s + w_c) \times \frac{r + r^2}{2} + 2.97C'\;(w_s + w_c) \times \frac{r + r^2}{2}$$

From the Elizabeth Bridge:

$$w_s = 3\,050 \text{ and } w_c = 5\,560\ldots\; w_s + w_c = 8\,610$$

then,

$$w'_l\,L' = 2r \times 3\,050 \times rC + 2rC \times 8\,610 \times \frac{r + r^2}{2}$$
$$+\; 2.97 \times rC \times 8\,610\,\frac{(r + r^2)}{2}$$

or,

$$w'_l\,L' = 6\,100\,r^2C + 8\,610\;Cr\;(r + r^2) + 12\,786\;Cr\;(r + r^2)\ldots\ldots(7)$$

From the Elizabeth Bridge layout, $C = 168$, and $L = 9.37 \times 168 = 1\,574$. From Equation (6):

$$w_lL = 2 \times 3\,050 \times 168 + 2 \times 168 \times 8\,610$$
$$+\; 2.97 \times 8\,610 \times 168 = 8\,214\,000 \text{ lb.}$$

Therefore,

$$w_l = \frac{8\,214\,000}{1\;574} = 5\,219 \text{ lb.}$$

Trying a 2 000-ft. total length:

$$r = \frac{2\,000}{1\,574} = 1.27\,;\; r^2 = 1.61\,;\text{ and, } r + r^2 = 2.88$$

Using Equation (7),

$$w'_l\,L' = 6\,100 \times 1.61 \times 168 + 8\,610 \times 168 \times 1.27 \times 2.88$$
$$+\; 12\,786 \times 168 \times 1.27 \times 2.88 = 14\,798\,000 \text{ lb.}$$

from which,

$$w'_l = \frac{14\,798\,000}{2\;000} = 7\,399$$

Similar calculations were made for several other total lengths of structure, and the results were plotted in Fig. 11, all points falling almost exactly on a right line.

Another right line on the same diagram gives the weights per linear foot for laterals. Combining these and adding 2 280 lb. for weight of metal in

the floor system gave the Class *A* line shown in Fig. 12, which includes also weights of metal on piers. Similar calculations determined the Class *B* line.

Fig. 13, compiled from the writer's office data, with considerable manipulation for additional cases, gives a similar record for the corresponding Type *A*

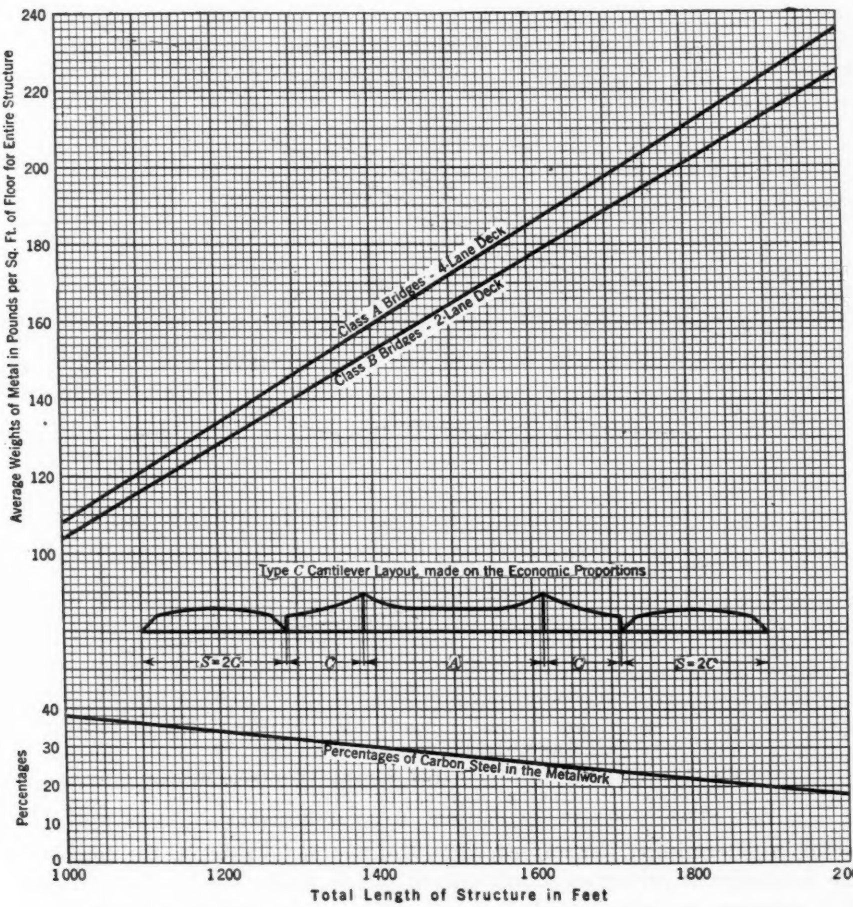

FIG. 12.—DIAGRAM OF WEIGHTS OF STEEL PER SQUARE FOOT OF FLOOR FOR CANTILEVER HIGHWAY BRIDGES OF TYPE *C*.

cantilever bridges, that type being the ordinary one of three spans, namely, two anchor arms, two cantilever arms, and a suspended span. (See Fig. 8.) These curves contain a proper allowance for weights of metal in anchorages and on piers.

In Fig. 14 are given the weights of metal per square foot of floor for simple-truss spans, built mainly of silicon steel; and in Fig. 15 are shown the corresponding values of similar bridges built entirely of carbon steel. In Figs. 14 and 15, if the structure has sidewalks, only one-half their area is to

be added to the area of the roadway when computing the total area of the floor. These curves do not apply to structures that carry electric railway tracks.

Figs. 14 and 15 will be found useful in determining the comparative economics of cantilever and simple-truss structures; they will also be found

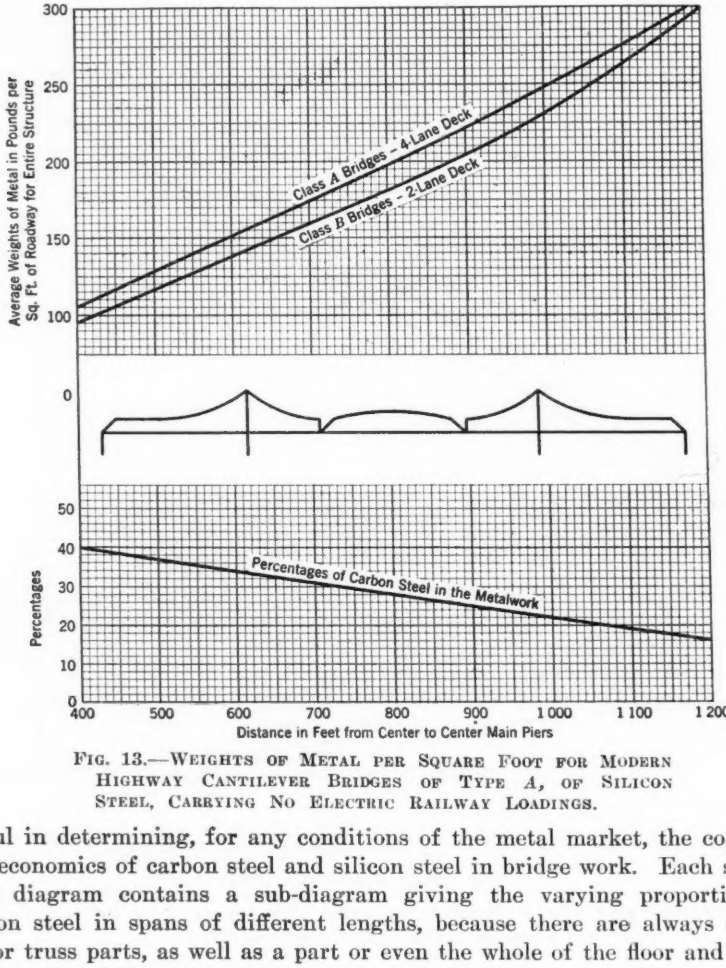

FIG. 13.—WEIGHTS OF METAL PER SQUARE FOOT FOR MODERN HIGHWAY CANTILEVER BRIDGES OF TYPE *A*, OF SILICON STEEL, CARRYING NO ELECTRIC RAILWAY LOADINGS.

useful in determining, for any conditions of the metal market, the comparative economics of carbon steel and silicon steel in bridge work. Each silicon-steel diagram contains a sub-diagram giving the varying proportions of carbon steel in spans of different lengths, because there are always certain minor truss parts, as well as a part or even the whole of the floor and lateral systems, in which carbon steel can be used advantageously.

The first of these two problems, that is, at what span length it pays to change from the simple-truss to the cantilever type, can be solved by a comparison of Figs. 13 and 14. Adopting the economic proportion, already established, for length of anchor arm equal to one-half the length of main opening, it is necessary to find from Fig. 13 the length, L, of the latter, for which the weight is the same as that of a simple-truss span of $\frac{2L}{3}$. Inspec-

tion shows that this condition exists for spans of 400 ft.; beyond that length the cantilever type will be economic.

As an example of determining the economics of silicon steel compared with carbon steel for simple-truss spans, assume that the former erected in a proposed Class *B* highway bridge is worth 7 cents per lb., as against 6 cents per lb. for the latter, and that the span length is 420 ft.

Fig. 15 shows that the weight of carbon steel per square foot is 164 lb., which, at 6 cents per lb., would cost \$9.84. Fig. 14 shows that the weight of metal per square foot is 140 lb., of which, 32%, or 45 lb., is carbon steel, leaving 95 lb. of silicon steel; that is,

45 lb., @ 6 cents	= \$2.70
95 lb., @ 7 cents	= 6.65
Total	= \$9.35

This shows a saving of 49 cents per sq. ft. of floor by adopting silicon steel.

If the span were 200 ft., as against 420 ft., what would be the economic comparison?

From Fig. 15,	
88 lb. @ 6 cents	= \$5.28
From Fig. 14,	
32 lb. @ 6 cents	= \$1.92
48 lb. @ 7 cents	= 3.36
Total	= \$5.28

This demonstration shows the costs to be exactly the same for the two metals in this span length.

As cantilever highway bridges have been shown to be uneconomic compared with a succession of equal-length, simple-truss spans, each less than 400 ft. long, the question of the comparative economics of the two steels will not arise for cantilever structures, because silicon steel will always be the cheaper for such long-span bridges; hence, no provision has been arranged for making the comparison.

The weights of metal per square foot of floor in Figs. 12, 13, 14, and 15 for Class *A* structures were computed for four-lane decks, and those for Class *B* structures for two-lane decks. To use them for any other lanes will require the following modifications:

Class *A* Structures:	
For a two-lane bridge add	6 per cent.
For a three-lane bridge add	2.05 per cent.
For a five-lane bridge deduct	2 per cent.
For a six-lane bridge deduct	4 per cent.
Class *B* Structures:	
For a three-lane bridge deduct	2.05 per cent.
For a four-lane bridge deduct	6 per cent.

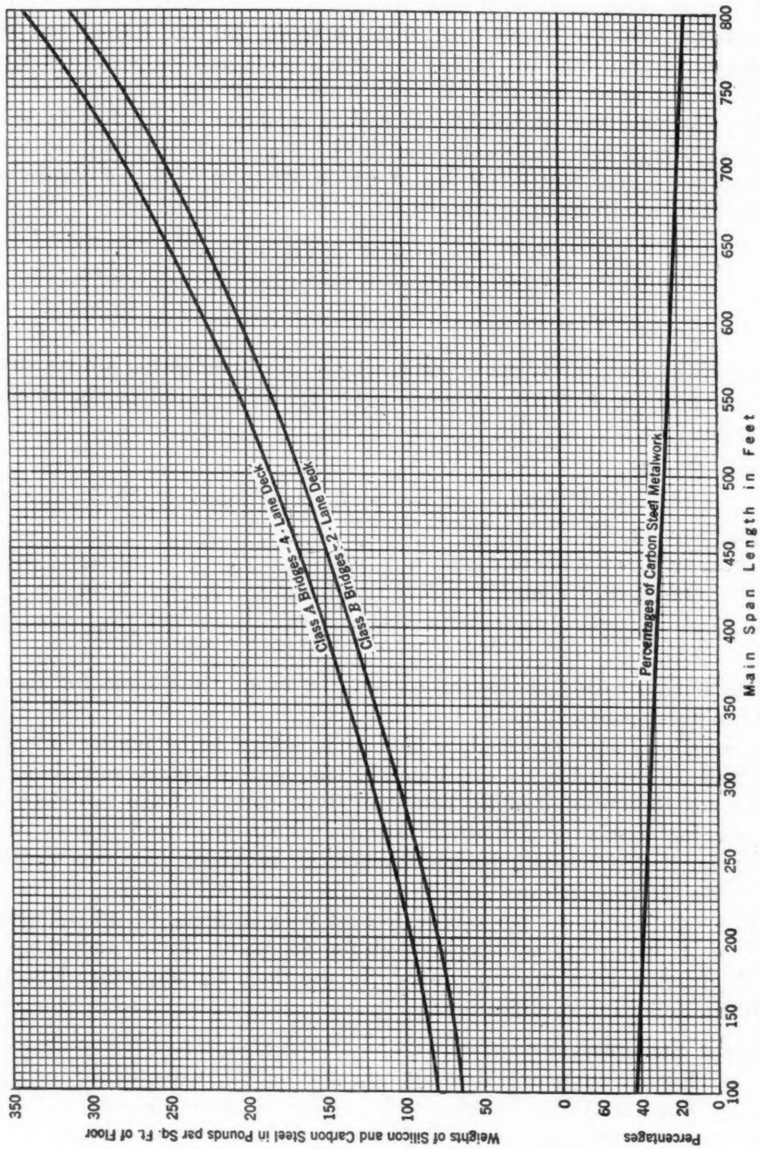

FIG. 14.—WEIGHTS OF METAL PER SQUARE FOOT OF FLOOR IN SIMPLE TRUSS HIGHWAY BRIDGES OF CARBON STEEL.

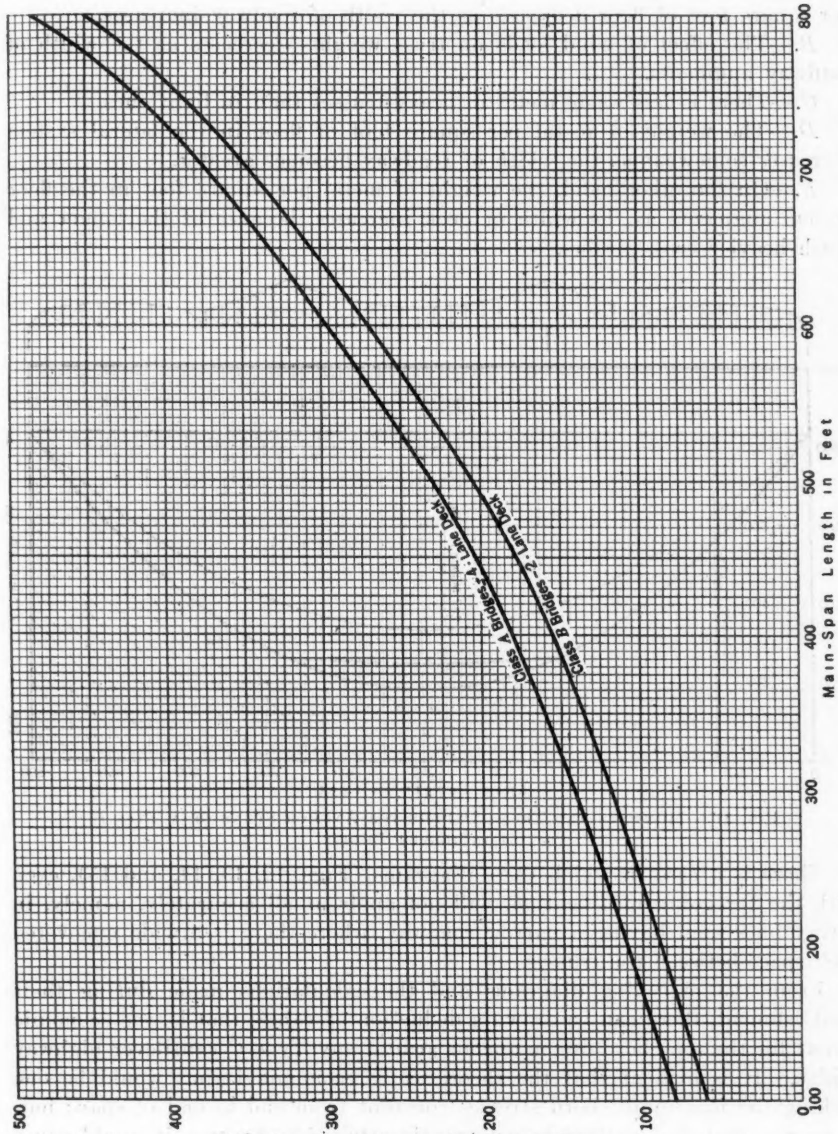

FIG. 15.—WEIGHTS OF METAL PER SQUARE FOOT OF FLOOR IN SIMPLE TRUSS HIGHWAY BRIDGES OF CARBON STEEL.

There are several reasons for this variation:

A.—According to the latest and most approved specifications, the live load per square foot of floor decreases as the width of roadway increases.

B.—The effect of wind loads on truss weights decreases as the width of roadway augments.

C.—There is less waste metal in heavy trusses than in light ones.

D.—The weight of metal per square foot of floor in the lateral system decreases somewhat as the width of roadway becomes greater.

E.—On the other hand, the weight of metal per square foot in the floor system augments as the width of deck increases, because of the longer and much heavier cross-girders.

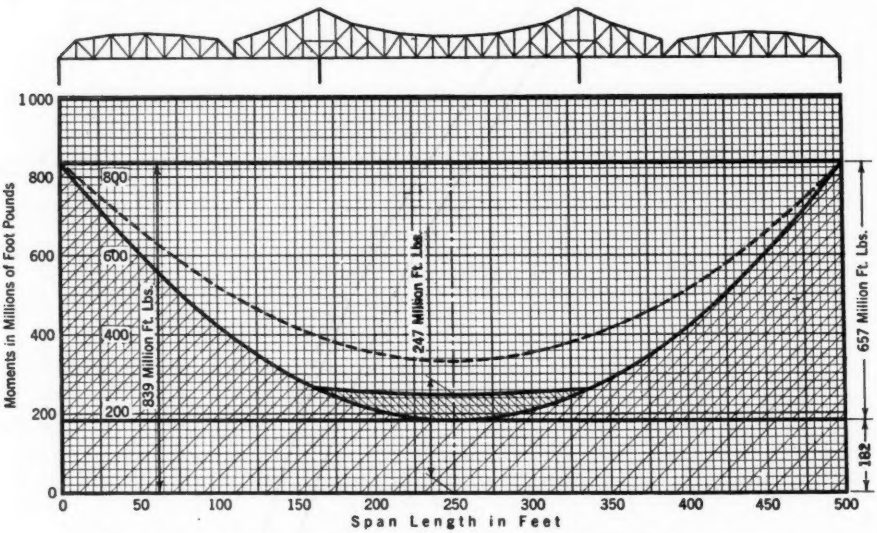

FIG. 16.—MOMENTS AND TRUSS DEPTHS FOR ANCHOR SPANS OF A TYPE *C* CANTILEVER BRIDGE.

There is a case in which these diagrams (Figs. 12, 13, 14, and 15), even with the foregoing modification, will not apply at all accurately, namely, in a very long span having a narrow roadway, where there is a wide uncovered deck space between the trusses.

From the preceding diagrams and the calculations made during their establishment, there can be drawn a deduction of value; that is, the economic layout for the top chord of the anchor arm of any Type *C* cantilever highway bridge. It is evident that the theoretically economic layout would be one making the maximum chord stresses constant from end to end of span; but, of course, that desideratum is not exactly attainable, because it would cause the top chord to be too irregular. However, it might be feasible to approximate it, as can be seen in Fig. 16, which represents the bending moments for a 500-ft. anchor span of a Type *C* cantilever bridge, 1 508 ft. long. This was prepared from the calculations used in determining some of the preceding results of this investigation.

It will be noted from Fig. 16 that, for any economic layout of a Type *C* cantilever highway bridge, the net dead-load bending moment at the middle of the anchor span is very small, in this case too small to be visible on the diagram.

Three different sets of hatchings will be observed: That for the parabola being for net dead-load moments; that of the rectangle being for extraneous live loading; and the small crescent representing one-half the net, direct, live-load bending moments. The ratio of compensated maximum bending moments at the center and ends is $\frac{247}{839} = 0.3$, nearly.

The economic truss depth over piers for this case is about 94 ft.; hence, a parabolic top chord with a central truss depth of 28 ft. would be the most economical; but it would be too shallow for both appearance and rigidity. Nothing less than four-tenths of the height over piers would be satisfactory, as can be seen by the small-scale layout of spans shown in Fig. 8. The ordinates of the dotted curve on the moment diagram, when compared with those of the top line of moments, will indicate how the sectional areas of chords will vary from uniformity when the truss depth at mid-span is four-tenths of that over piers.

In the preparation of the calculations for this paper a fact was established from the voluminous records of the writer's firm that is worthy of chronicling, namely, that, for the trusses of an entire cantilever bridge, the weight of metal divides itself between chords and webs in the percentage proportion of 60 to 40; and this distribution applies quite closely to each of the main divisions of the structure, that is, anchor arms, cantilever arms, suspended span, and anchor span. In determining this proportion, the inclined end posts were assumed to belong to the web, and the gusset-plate weights were equally divided between web and chords.

Résumé of Results

The findings of this research are, as follows:

First.—The economic ratio of length of suspended span to length of main opening in an ordinary three-span cantilever highway bridge varies from 0.45 to 0.50.

Second.—The economic truss depth over main piers in that type of structure varies from 12 to 15% of the distance between centers of main piers, with an average of 13.5 per cent.

Third.—The economic length for the anchor arms in that type of structure, when the distance between centers of main piers is fixed, is as short as the æsthetic appearance of the bridge will permit, say 25 or 30% of the main opening.

Fourth.—When, for the same type of layout, the distance between anchor piers is fixed, with no restriction as to the locations of the two main piers, the economic length for each anchor arm is about one-quarter of the total length of structure.

Fifth.—In a highway bridge having at one end a suspended span of length, l, a cantilever arm of one-half that length, and an adjoining anchor span of a length, y, that may be varied, the ratio of truss weights per linear foot for the spans, y and l, are as follows:

Values of $\frac{y}{l}$	Ratio of truss weights	Values of $\frac{y}{l}$	Ratio of truss weights
0.8	2.77	1.6	1.77
1.0	2.47	1.8	1.74
1.2	2.20	2.0	2.63
1.4	1.95		

Sixth.—In a structure consisting of a suspended span, a cantilever arm, an anchor span, a like cantilever arm, and a like suspended span, the economic length of the anchor span is about 36% of the total length of the layout; and even a small variation from this economic proportion involves a material excess in the total weight of truss metal.

Seventh.—In highway bridges it is economic to change to cantilever construction from three simple truss spans of equal length, when that length is about 400 ft.

Eighth.—An ordinary current unit prices for metals erected, in simple-truss spans it begins to be economic to use silicon steel at a span length of about 200 ft. for four-lane bridges and 300 ft. for two-lane bridges; but, for truly warranted, cantilever structures, silicon steel will invariably be found to be less expensive than carbon steel.

Ninth.—The economic truss depth at mid-length of an anchor span of economic proportions in a Type *C* cantilever highway bridge is about three-tenths of that over the piers; but such a depth would be too small for both appearance and rigidity. This inferior limit should be about four-tenths of the depth over piers.

Tenth.—In the trusses of highway cantilever bridges, the weight of metal generally divides itself between chords and web in the ratio of 3 to 2; and this division applies to the different spans as well as to the structure as a whole.

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PAPERS

DESIGN CHARACTERISTICS OF THE READING OVERBUILD TRANSMISSION LINE

By Frederick W. Deck,[1] Assoc. M. Am. Soc. C. E.

Synopsis

The usual type of transmission-line structure that supports high-tension lines across open country is of so frequent occurrence, that little comment is aroused when such a line is constructed, unless it is of unusual length or carrying capacity. Sometimes, however, the approach of a power line to the boundaries of a city requires the use of unusual types of supporting structures in the effort to minimize right-of-way costs and provide additional safety in regions where the damage attendant upon a failure of the line would be disastrous.

This was the case at Philadelphia, Pa., where the power from the Conowingo Hydro-Electric Plant and interconnection lines is brought into the city on wires supported by structures built over the tracks of the Reading Railroad. Certain features of the structural designs are most unusual and interesting, and are briefly described in this paper.

General Description of Project

Power generated at the hydro-electric plant[2] of the Philadelphia Electric Company, at Conowingo, Md., on the Susquehanna River, is transmitted about 57 miles at 220 kv. to the large distributing sub-station at Plymouth Meeting, Pa., about 10 miles outside of Philadelphia. This sub-station is also the terminus of 220-kv. lines connecting to adjacent large systems in Pennsylvania and New Jersey. From Plymouth Meeting, the power is carried at 66 kv. to sub-stations within the city.

Note.—Written discussion on this paper will be closed in May, 1932, *Proceedings*.

[1] Structural Engr., Mech. Div., Philadelphia Elec. Co., Philadelphia, Pa.

[2] For description of this plant, see *Transactions*, Am. Soc. C. E., Vol. 93 (1929), p. 970.

For this latter section of the power-transmission scheme it was desirable to provide avenues of entry to the city as independent of each other and as capable of expansion as possible. Over a part of the distance from Plymouth Meeting to Philadelphia, this was accomplished by building parallel tower lines carrying two circuits each, on a wide, corporately owned, right of way. On account of the closely settled condition of the district within the environs of the city, and the consequent high cost of right of way, the transmission lines were brought in over two separate railroad rights of way. A co-operative agreement between the Electric Company and the Railroad Company on the basis of mutual benefit, permitted the adoption of this method of transmission within the city limits.

One of these lines was built in 1928, with two circuits in operation.[3] It is the purpose of this paper to describe the design of the second of these lines, about $4\frac{1}{2}$ miles in length, embodying unusual features in the design of the transmission structures and their foundations. A similar problem of overbuild or collinear construction was solved in the first overbuild line; but in the line described herein, the difficulties of design and construction were far greater than those encountered in the earlier work.

Layout of Transmission Line

This transmission line is built over and along a passenger railroad, through a city street with buildings on each side, and with many of its supporting structures located on a concrete and steel viaduct (see Fig. 1). In general,

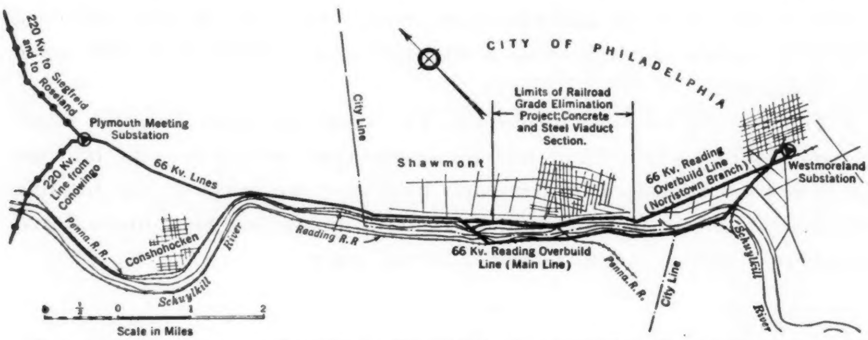

Fig. 1.—Routes of the Conowingo Transmission Lines from Plymouth Meeting to Philadelphia, Pa.

the limitations of the right of way made necessary the use of some type of structure occupying a minimum of area at the supports, while carrying the power wires as nearly as possible over the center of the trackage, in order to provide adequate lateral clearance from the boundaries of the railroad right of way.

One section of the railroad lies on a tangent, but the remainder of the line curves through a city street; and in this latter part the Railroad Company rebuilt its surface line as an elevated section and eliminated grade crossings.

[3] *Engineering News-Record*, May 3, 1928, p. 686.

The problem of layout, therefore, involved first, the determination of the most economical span length between structures, providing adequate vertical clearance above the rails and lateral clearance for the side swing of the wires, while using a tension in the wires that would allow an economical tower design. In order to obtain this economical structure and maintain the required vertical clearance, it was necessary to reach a balance between the low-wire tension usable with high structures and a high-wire tension usable with low structures. For the straight line, a basic span of 600 ft. was chosen, dependent on the previously mentioned considerations of vertical and lateral clearance, and economical structure design. The actual maximum span used was 803 ft., and the minimum, 280 ft.

On curves the structures were located with shorter intervening spans; and over the elevated viaduct section the transmission-line construction was restricted as closely as possible to the width actually used by the two-track viaduct. This restriction forced the incorporation of the footings for the transmission structures as a part of the railroad viaduct structure. In this section, also, it was not always possible to locate the structures where they would be most suitable from the theoretical layout point of view, and frequently the locations were those at which the supports for the power wires would interfere least with the railroad facilities, which were already restricted.

General Description of Structures

The original estimates of cost for supporting structures were based on a more or less hypothetical bridge type of structure, with vertical sides, spanning the entire number of tracks at each location. Exhaustive field and office studies of the conditions involved, aided by surveys of the right of way, buildings, and other obstacles, resulted in the development of the rather unusual types of structures described herein.

While uniformity in shape and design was a primary consideration in the development of supports for this line, the location and surroundings of each of the structures speedily eliminated absolute uniformity in type and finally resulted in the evolution and selection of seven different types of transmission-line structures including the terminal structure of the overbuild section, which is a two-circuit tower. In each case the aim was to devise a structure which would (1) support the heavy wire loads, (2) give the clearances required, and (3) afford a maximum simplicity of design and ease of construction, all at a minimum cost.

The height of all structures was determined by the required clearance of wires at the lowest point of the sag curve from the rail, and a standard height of approximately 68 ft. above the rail for the lowest cross-arm on bridge structures and towers was adopted. Where greater clearance was required in the adjacent spans for crossing over other railroads, or streets, this height was increased to 78 ft., or to the maximum of 90.5 ft.

A vertical clearance of 45 ft. from wires to rail, with the maximum sag in the wires under normal weather conditions, is maintained throughout, with a minimum diagonal clearance of 20 ft. between wires and the nearest portion of any building.

The railroad right of way for a large part of the line is 66 ft. wide and, where possible, 65.5 ft. was adopted as the over-all width of bridge structures, giving space for four tracks, ample clearance on each side of the outside track, and a framed post, 3 ft. 3 in., in width outside the clearance line on each side (see Fig. 2). Modification of this width was required at points

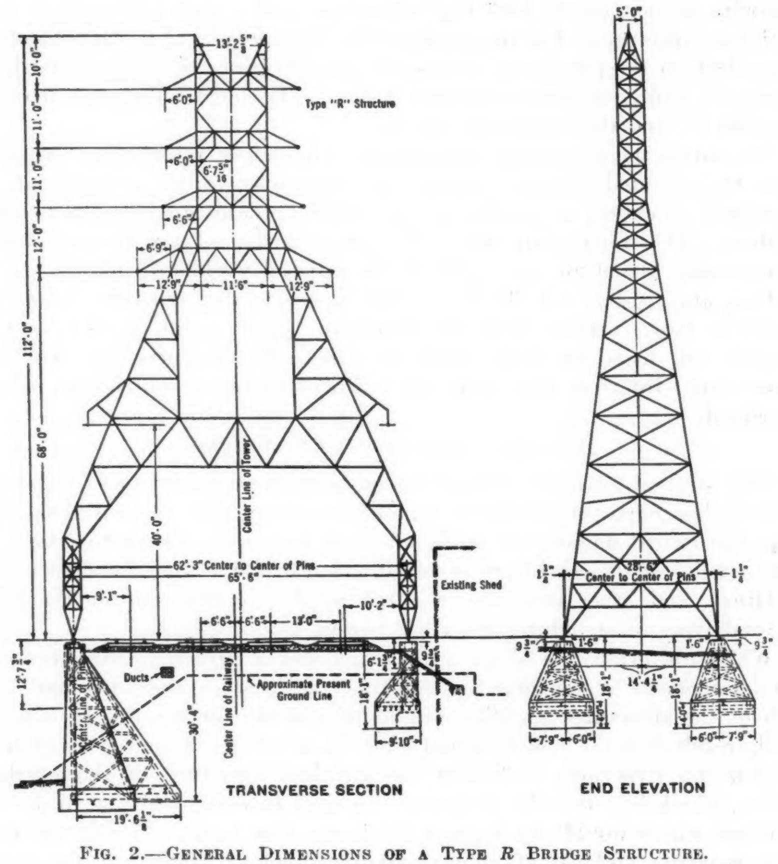

FIG. 2.—GENERAL DIMENSIONS OF A TYPE *R* BRIDGE STRUCTURE.

where structures were located along and over the elevated viaduct section of the railroad, and elsewhere, as demanded by right-of-way conditions. Typical dimensions are shown in Fig. 2.

The cross-arm arrangement for carrying wires was determined by the required lateral and vertical clearances between conductors, and by the height, width, and slope of the sides of the structures. In general, there are two types of tops, one comprising the tower and the eccentric bridge structure, and the other (see Fig. 2) covering the remaining four types. The design of these types was based on the desirability of maintaining certain features of symmetry in loading with a minimum of change in the slope of the posts.

Many changes from the original plans were made as the result of the development of unforeseen features. The early construction of the elevated viaduct section of the railroad to eliminate grade crossings had the effect of forcing the completion of the designs of most of the foundations before the design of the structures was finished, since it was desirable to incorporate this work in the course of construction simultaneously with the railroad work rather than tear down and rebuild later.

The necessity of providing for two single-phase transmission circuits for railroad use caused a revision of the shapes and sizes of structures different from those used on the first line. Four cross-arms carrying thirteen conductor wires were required as against three cross-arms carrying twelve conductor wires used on the other overbuild line mentioned previously.[3] These and many other revisions were accomplished, and the design of all the structures was completed, resulting in the use of forty units on this line.

Foundation Problem

In such construction within the narrow confines of the railroad right of way, the foundations presented a major problem in design. To select those foundation shapes which would bear, with unquestionable security, the heavy loads transmitted from the superstructures above, required the utmost care in maintaining a nice balance between the complexities in design and consequent cost, and the lavish use of labor and materials in a simpler design. In some places, however, this problem was simplified in that the exigencies of the situation demanded both an unusual design and the use of much concrete.

Arrangement of Wires and Their Characteristics

On all structures, the power-transmission wires of the Electric Company are arranged in three circuits, parallel to each other, with phases vertical. The two ground wires are carried above, and the two single-phase transmission circuits of the Railroad Company are carried on the lowest cross-arm, the four wires being arranged abreast of each other. Two or more feeder wires for the railroad electrification are carried on brackets attached to the sides of the structures about 40 ft. above the track level. Provision was made on all "bridge" structures for suspending the catenary type of electrification wires of the Railroad Company from points on the inside of the lower arch of the structure. On tower structures, the catenary wires are supported from a cross-member one end of which is attached to a pole on the other side of the tracks, and the other end to the tower.

The conductor wires for the Electric Company are 500 000 cir. mils, bare-stranded, hard-drawn copper, with an ultimate strength of 23 450 lb., and an allowable working tension of 11 770 lb. They are strung so as to have a maximum tension of 8 000 lb. under the maximum loading of $\frac{1}{2}$ in. of ice measured radially, and 8 lb. of wind per sq. ft. of longitudinal cross-section of the loaded wire exposed to the wind. The ground wires are a composite cable consisting of seven strands of a copper, aluminum, and tin alloy, and an outer layer of twelve strands of a cadmium-copper alloy. This cable has

a diameter of 0.477 in., with a breaking strength of 12 500 lb. and an elastic limit of 7 500 lb. Under maximum loading of $\frac{1}{2}$ in. of radial ice, and 8 lb. of wind per sq. ft., it has a maximum tension of 3 900 lb., and thereby closely conforms in sag to the conductor wires.

The conductor wires for the Reading Railroad are smaller than those for the Electric Company. The other wires supported by the structures described are for the purpose desired by the Railroad Company in its electrification plan. All the wires used for the Electric Company are attached to the cross-arms of the supporting structures through strain-insulator assemblies of a double string of nine standard strength insulator units, with a total ultimate strength of the assembly of 24 000 lb.

The pull of the conductors is thus applied directly to the cross-arms, imposing a large load on the structures. This method of attachment was chosen because it afforded advantages in clearance, a saving in the heights of structures, and additional security of the line against failure in this important section over the railroad.

Detailed Description of Structure

The structures have been classified into seven types—*R, T, U, V,* and *W* bridges and *M* and *C* towers, respectively, and, for convenience, these designations will be used in the paper.

Type R Bridge.—The bridge structure shown in Fig. 2 is 65.5 ft. wide at the base in the transverse view; it spans four tracks, is 68 ft. high to the

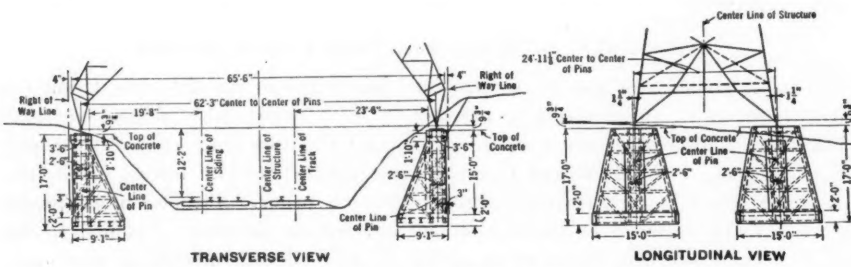

Fig. 3.—Modified End Posts of a Type *R* Bridge Structure.

lowest cross-arm, and 112 ft. high over all. This is the type used most frequently on this work. It is a symmetrical structure, the lowest section being an arch portal, and framed throughout. The posts are pin-connected to the footings through standard cast-steel shoes which are secured to the upper section of the steel framework of the footings by anchor-bolts.

This structure was modified as shown in Fig. 3, to enable it to meet the conditions of high banks on both sides of the tracks without excessive excavation, by cutting off the lower posts at different levels. The cross-arms are maintained at the standard height of 68 ft. above the rail, and the structure is the same as that shown in Fig. 2, except for the shortened legs. This innovation sometimes resulted in a two-hinged arch, with hinges at different levels.

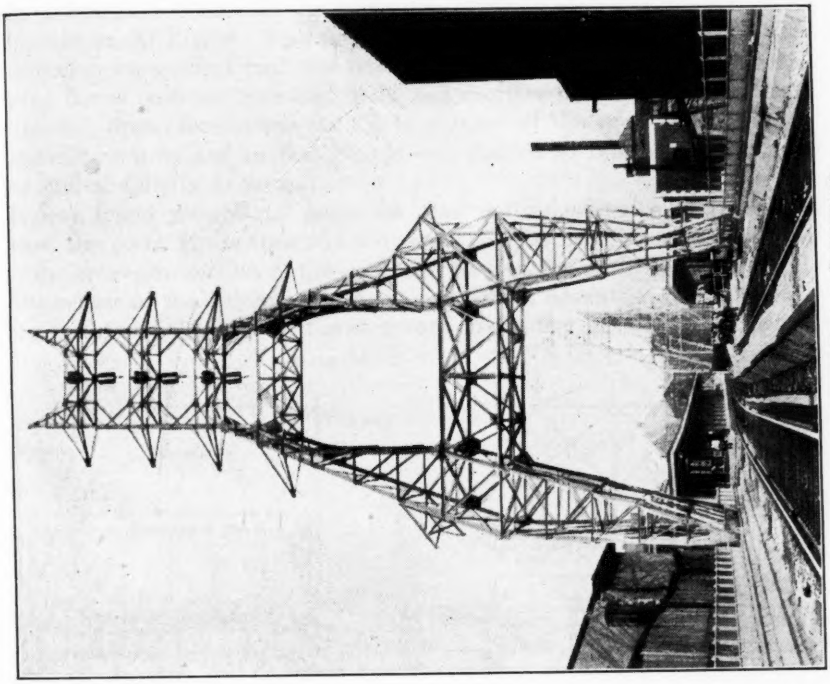

FIG. 5.—TYPE *U* BRIDGE STRUCTURE.

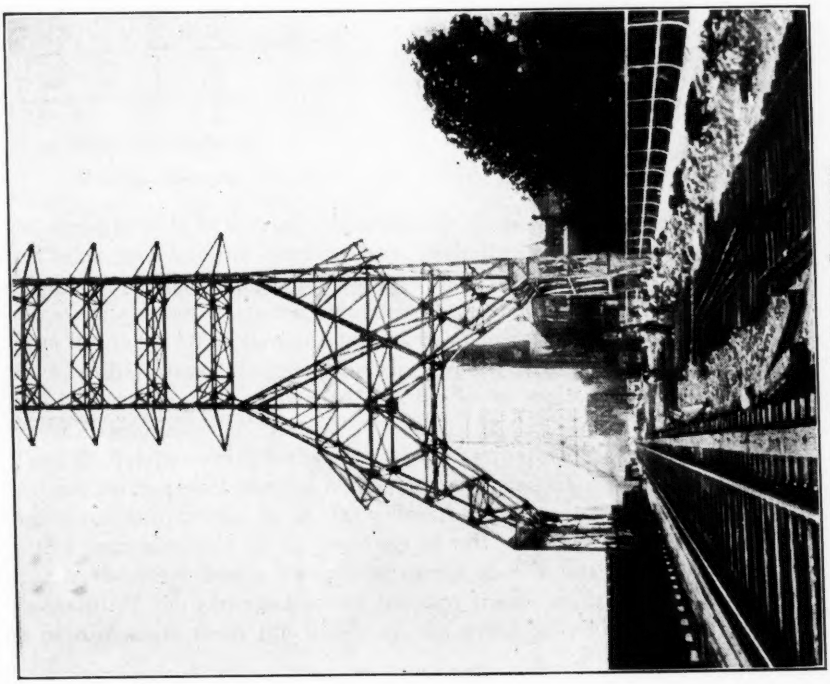

FIG. 4.—TYPE *T* BRIDGE STRUCTURE.

Type T Bridge.—Possibly the most interesting of all the bridge structures is that shown in Fig. 4. It was designed for the purpose of carrying the transmission wires offset from the center of the structure, in order to remove them as far as possible from high buildings or other obstacles on one side of the tracks. Space limitations for the base required the use of some kind of eccentric structure, and in this case it was desired to retain as much symmetry and simplicity as possible.

It was found possible to make the base width just twice that required between the posts of the standard cross-arm, and by carrying one post down from the cross-arm section to the ground vertically, the other post was exactly in the center of the structure, with the consequent advantage in distribution of the load from this side of the structure to the two post bases. The load

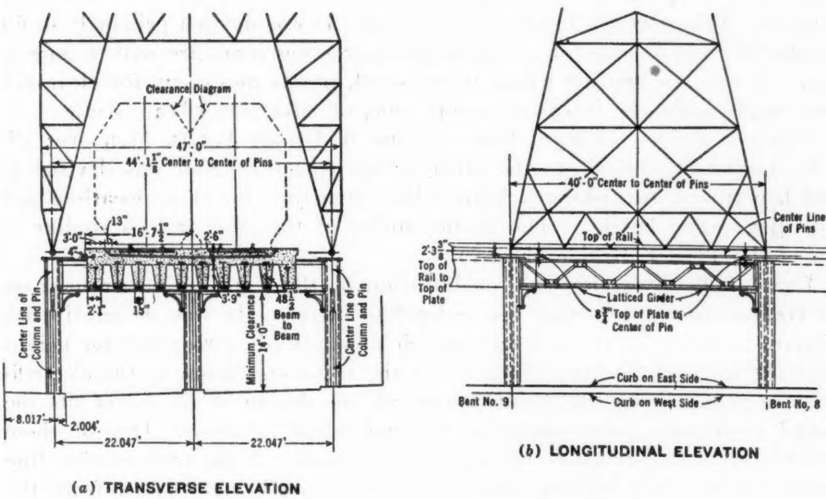

FIG. 6.—GENERAL DIMENSIONS OF A TYPE *U* BRIDGE STRUCTURE.

on the straight side is carried down mainly through the vertical post to the pin. The various loading combinations, including torsional effects, naturally modified this to some extent, but the results of this symmetry of arrangement where possible in an eccentric structure were shown in its consequent comparative lightness. The structure spans two tracks; it is pin-connected to the footings in the manner described for the Type *R* Bridge; framed throughout; and, as in the case of the other bridges, the lower section is an arch portal in its stress diagnosis.

Type U Bridge.—Fig. 5 shows a type of structure that was arranged for special use on the steel portion of the railroad viaduct. It is symmetrical in shape, spans two tracks, is 44 ft. wide from center to center of pins, and measures approximately 68 ft. from top of rail to the lowest cross-arm. It is similar to the other bridge structures, except that it was possible in this case (on account of the arrangement of the two tracks spanned with their clearance requirements from the inside of the arch) to make the framed posts

slope so that in the transverse view, a straight line drawn through the posts from cross-arms to pins will always lie within the width of the framed post section. This naturally aided the design and the resulting structure is one of the lightest used on the entire line.

In the longitudinal view it covered a somewhat greater span than would be usual in a tower of this type (see Fig. 6(*b*)). The length of 40 ft. between pins was used to allow them to rest directly above the posts of the steel viaduct, thus reducing the complications in loading. This structure is attached directly to the steel of the railroad viaduct and, therefore, requires no separate footings. A standard shoe was used.

The Type V Bridge.—The structure shown in Fig. 7 is similar to those shown in Figs. 2, 4, and 5. Due to clearance requirements the inside outline of the steel arch does not follow the same lines as those of the simpled *U*-type (Fig. 5). This tower is 122 ft. high over all. It was devised primarily to fill the special requirements of a high railroad-crossing structure with a narrow base. In the one location where it was used, it was necessary, for clearance purposes, to make the two posts on one side, of solid girder-beam sections.

Type W Bridge.—Fig. 8 shows a Type *W* Bridge 112 ft. high over all. It is similar in analysis to the other bridge structures and was devised to span two tracks, furnishing a narrow base structure for those locations not requiring special height. The inside outline of the steel arch is similar to that of the Type *V* Bridge (Fig. 7).

Type M Tower.—This is a modification of the Type *H* Tower used on the transmission line across the Schuylkill River.[4] It was designed with standard sections 7.5 ft. in height, which were placed as required for height above the level of the upper support for the transverse beam of the electrification system. From this point downward, the design of the tower fits the ground conditions which obtain at the individual locations. One of these towers used as the structure to support the crossing of the transmission line over a street, is the highest structure on the line, being 90.5 ft. from the lowest cross-arm to the top of rail. This type of structure was used where right-of-way conditions permitted, or where they required the location of the tower on one side of the tracks. (See Fig. 9.)

Type C Tower.—The standard, open-country, angle tower, Type *C*, carries two circuits only; the sides are symmetrical on all faces; and the structure is designed to stand the pull of the wires when there is a change of 60° in the direction of the line at that tower. This is the only two-circuit tower on the line, and carries the wires of the Electric Company from the last tower on the railroad right of way to the cross-country line from Plymouth Meeting.

Basis of Design

These structures are all planned for the usual longitudinal, transverse, vertical, and torsional loading effects encountered in tower design. The weight of wires loaded with ice, or unloaded, the wind on the wires loaded or unloaded, the weight of the tower itself, the weight of insulators and hardware, the transverse pull at the wire supports due to deflection angles in the

[4] *Engineering News-Record*, May 3, 1928, p. 689, Fig. 5.

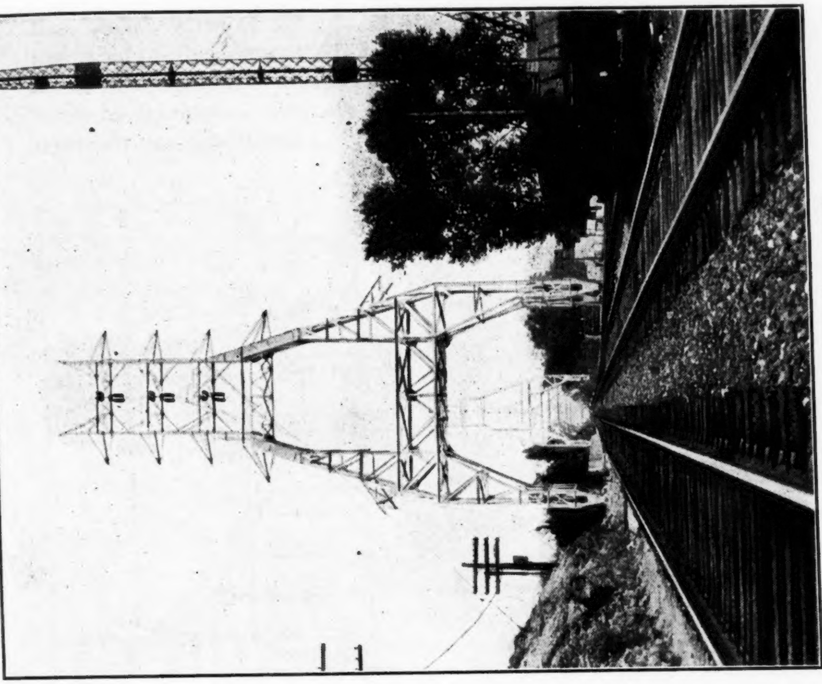

FIG. 8.—TYPE W BRIDGE STRUCTURE.

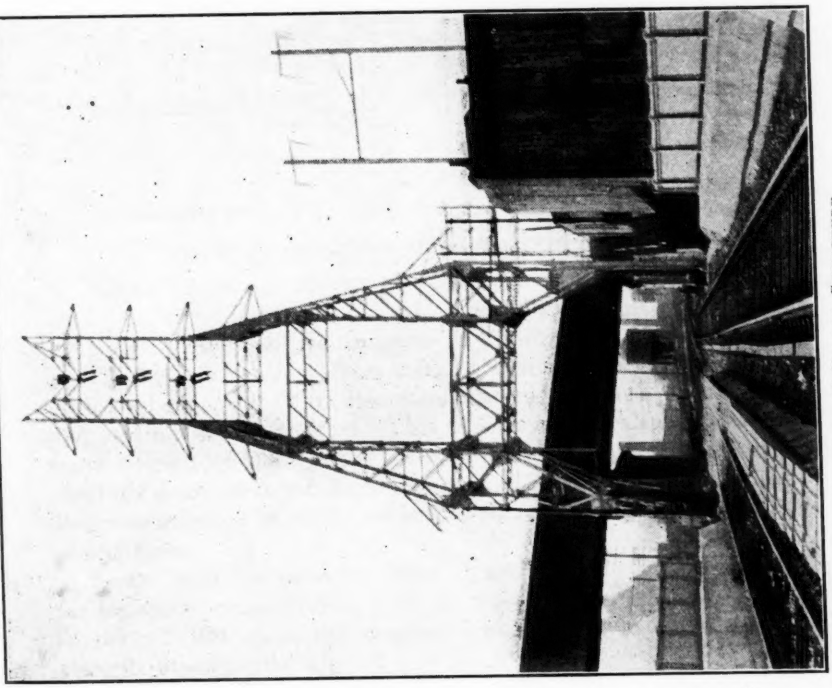

FIG. 7.—TYPE V BRIDGE STRUCTURE.

line, broken wires of the number specified in the various loading conditions, and wind on the tower itself, are among the factors which were considered in the several combinations, in order to produce a design from which a structure could be fabricated, adequate to meet all the calls on its strength under the most adverse conditions.

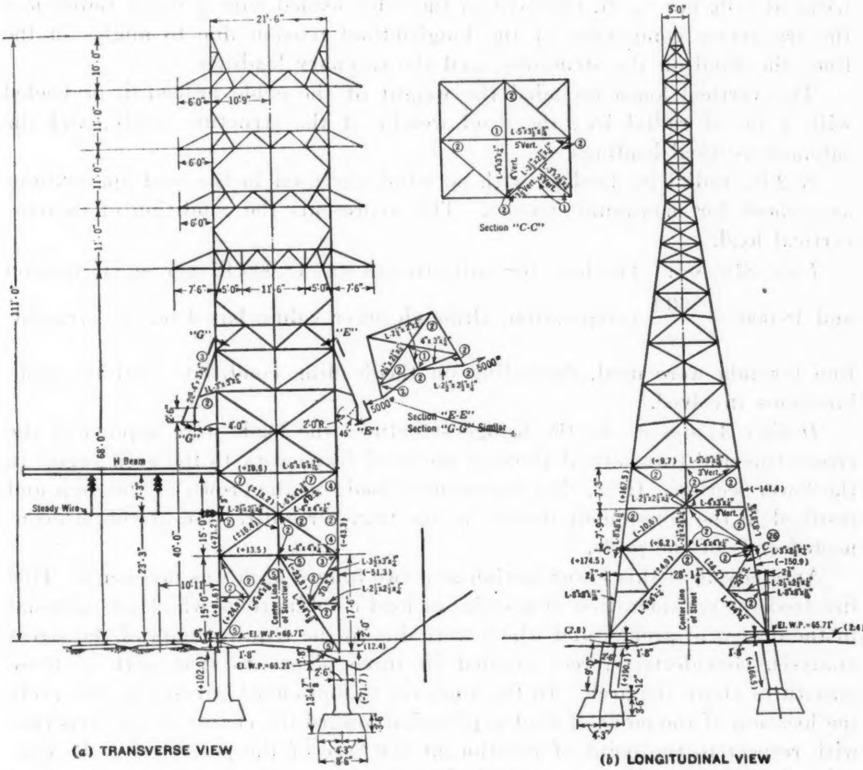

FIG. 9.—GENERAL DIMENSIONS OF A TYPE *M* TOWER STRUCTURE.

Design Loads.—A tension of 8 000 lb. in the wire occurring at 0° Fahr., with $\frac{1}{2}$ in. of radial ice on the cable and 8 lb. per sq. ft. of wind load applied to it, formed the base requirements of loading.

The 8 000-lb. tension allows a sag of approximately 15 ft. on a 600-ft. span with level supports. With this base, sags and tensions for other temperatures and loadings were computed. This tension is equivalent to one-half the ultimate strength of the wire for a 600-ft. span with a 10-ft. sag for the 500 000 cir. mils, bare, stranded, hard-drawn, copper cable used. Wind on the structure was assumed at 13 lb. per sq. ft. on twice the projected exposed surface of one face.

Loads from the catenary wires comprised a 10 000-lb., longitudinal pull for broken wire conditions; 3 600 lb. for transverse loading at each catenary support; 10 000 lb. at the steadier wire support; and a vertical loading of 1 800 lb. at each wire support.

Conditions for loading the tower included the longitudinal pull of nine conductors of the Electric Company at 8 000 lb. each, and four conductors of the Railroad Company at 5 000 lb. each; or 11 770 lb. each at any four of the eleven cable supports, or 7 500 lb. each at the four lower conductors.

The transverse loads on the structure consisted of the wind load on the wires at 8 lb. per sq. ft. effective on the cable loaded with $\frac{1}{2}$ in. of radial ice; the transverse component of the longitudinal tension due to angles in the line; the wind on the structure; and the catenary loadings.

The vertical loads included the weight of the cable unloaded, or loaded with $\frac{1}{2}$ in. of radial ice; the dead weight of the structure itself; and the catenary vertical loadings.

A 2-in. radial ice loading with no wind was used in the load applications as a check for maximum stresses. This represents the condition of extreme vertical load.

Unit Stresses.—The base for unit stresses was 18 000 lb. per sq. in. tension and $18\,000 - \frac{60\,l}{r}$ compression, although other values based on the straight-line formula were used, depending on the loading conditions and the combinations involved.

Design Analysis.—In the bridge structures the loads were applied at the cross-arms and transmitted through the steel framework to the arch portal in the lower section. Here, they represented loads on the crown of the arch and resulted in the consequent thrusts in the transverse direction at the pin-connected ends of the posts.

An analysis of this lower section as a two-hinged arch was necessary. This involved the consideration of a series of load combinations which are unusual in the design of arches, and which were due to the requirements of the stress analysis, considering forces applied in three planes to that part of these structures above the arch. In the analysis, to find chord stresses in this arch, the location of the point of load application toward the center of the structure with respect to the point of reaction at the base of the posts had to be considered, not only in the vertical direction, but also in the longitudinal direction.

The unequal length of the legs of one of the types further complicated this problem of arch design. The transverse horizontal arch reactions were increased considerably with the shortening of the posts.

General Description of Foundations

In the course of the preliminary design of the structures it was necessary to consider the co-ordination of the construction work on some of the foundations with the improvements on the new railroad viaduct. Reactions for the superstructures were determined, and the footings designed for all the structures that were involved in the railroad program for elimination of grade crossings.

It was the original intention to adopt a uniform design of footings throughout the line, as far as possible. The policy of maintaining uniformity

in design was followed as closely as possible throughout, but in this overbuild of the masonry and steel viaduct section of the railroad, it was found necessary to locate the foundations in masonry walls, in fresh earth-fills, and as integral parts of the framing of the steel viaduct; and these restrictions of location rendered absolute uniformity impossible. It was usually found more economical to design a special footing for the case in question than to extend the so-called standard to cover the requirements at that location.

Footings that formed integral parts of the retaining walls, or other railroad construction, were placed as the walls were built. Expansion joints were eliminated as often as possible from the short section of wall that contained the footings.

In order to remain within the right-of-way boundaries of the railroad, and still provide adequate support for the superstructures at points very close to the right-of-way lines, most of the footings were necessarily eccentric in shape in the transverse view.

All these footings were designed to support the superstructure loads, not only under its normal dead-weight condition, but also under the loadings imposed by extreme weather conditions or unusual, unbalanced wire pulls. Thus, they were called upon to withstand direct down thrust and uplift with or without horizontal, longitudinal, or transverse shear at the top of the foundation. The adoption of pin connections in the case of the bridge type of structures between the superstructure framework and the footing frame, eliminated the bending in the transverse direction at the top of the footing, which would otherwise have been present due to the loading of the wide transverse arch. The transverse and longitudinal shear, down thrust, and uplift were cared for as described for each of the several types, maintaining the unit pressures on the soil. concrete, or other supporting medium, within very conservative limits.

It will be noted that in all the foundations described herein, a steel framework was provided, which was structurally competent to sustain all the superstructure loads placed upon it, independent of the concrete in which it was invariably encased. Actually, the bond existent between the steel and the surrounding concrete is active throughout the footing in the distribution of stress. The use of a steel frame instead of reinforced concrete footings was deemed to be safer, easier to construct in accordance with the office design, and more economical, in these cases of unusual loading.

The footing as a unit—concrete and steel—was always designed to sustain all loads, including those due to the tower, retaining wall, train loads, or other influences that could act upon it. The steel was designed to distribute the loads from the tower throughout the footing, and such reinforcement as was required was placed to bond the concrete where the steel frame was inadequate for this purpose.

Soil Conditions

Borings had been taken in many cases, and these, together with contemporary local construction experience, were found of assistance in the footing design. It was impossible to rely entirely upon these aids, however, since soil conditions were often found to vary with extreme abruptness. About 2½ miles

of this line over the railroad is located along a hillside, where the slope is transverse to the line toward the Schuylkill River (see Fig. 1). The rock stratum frequently is outcrop and, at other times it is just a few feet below the natural surface. It is traversed, transversely, by numerous dips and fissures toward the river, and some of these, being well covered by old fill, were not discovered until the foundation excavation was well advanced. Several times these depressions were found to be old stream beds, and then piles or concrete piers were used to support the walls and foundation above. In general, the soil was of such character as to allow the use of 4 000 lb. per sq. ft. as a bearing value, a base conforming to the unit stresses allowable in the steel when the structure was subjected to the normal heavy loading condition. An additional factor of 10%, or more, was added to the post reactions of the structures in computing the loads to be applied to the foundations.

Detailed Description of Foundations

For the Type *T* bridge structure with its eccentric arrangement (Fig. 4), and also for the Type *R* bridge (Fig. 2), the footing devised for earth conditions and as the basic design for use where applicable, is shown in Figs. 2 and 3. The bearing on the soil is eccentric, and the assistance of lateral passive earth pressure was used to great advantage in producing an economical design. Longitudinally, the footing was designed so that the axis of pressure from the post of the tower would pass through the center of the bearing base, thereby distributing the down thrust more equally in the longitudinal direction. This type of footing can be used where the ground is level, or in banks sloping to either side of the footing in the transverse direction. It is capable of extension to provide additional depth needed for resistance to down thrust, uplift, or transverse forces. The vertical inside face, to a depth of about 6 ft. below the rail level, allows the placing of ducts for signal lines and communication cables which otherwise would have been forced nearer the tracks into an undesirable position of proximity to the rail movements. For some cases of extension it was found possible to place a block on the inner side, below the normal base line, which acted to increase the resistance to uplift and lateral thrust.

These footings (Fig. 10), when placed within a concrete retaining wall as illustrated in Fig. 11, were designed as a part of the wall, in addition to their individual analysis. The wall was first investigated for bearing, overturning, and sliding under normal conditions, without the loads due to the transmission-line structures. The additional loads were then applied, and it was determined whether this addition required more strength or bearing surface than was existent in the wall itself. If so, the wall was changed in section to accommodate the requirements at that location. Usually, the steel structure of the footing was carried down near the base of the wall, allowing but little "spreading" of the effects of the loads applied through it. When the steel was at some distance above the base, it was assumed that the spreading of the load effect downward in the longitudinal direction through the concrete of the wall occurred at an angle of 45° with the vertical, thereby decreasing the unit pressure on the soil at the base of the wall.

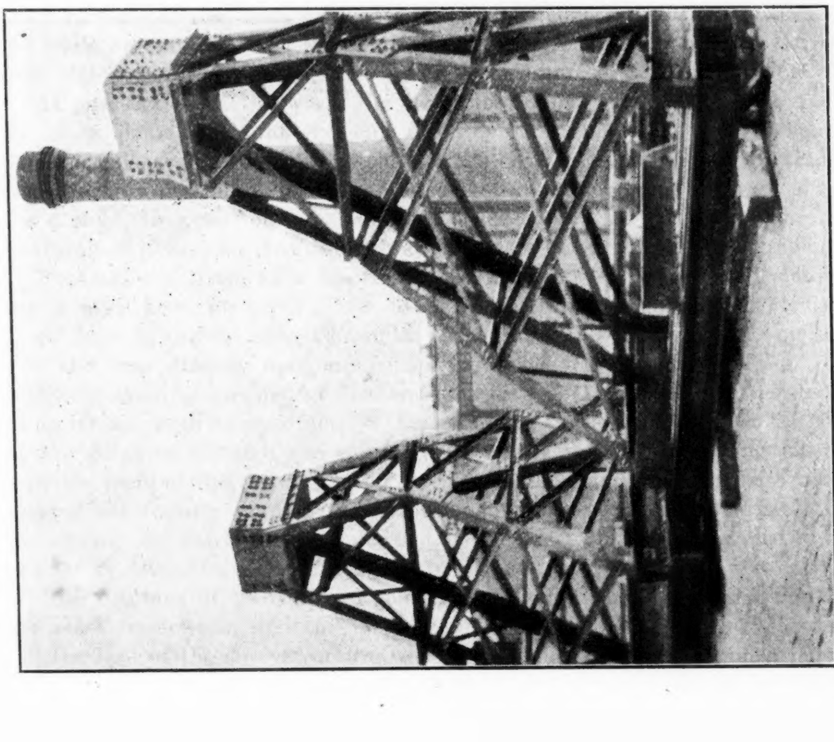

FIG. 10.—STEEL FOOTINGS BEFORE BEING ENCASED IN CONCRETE.

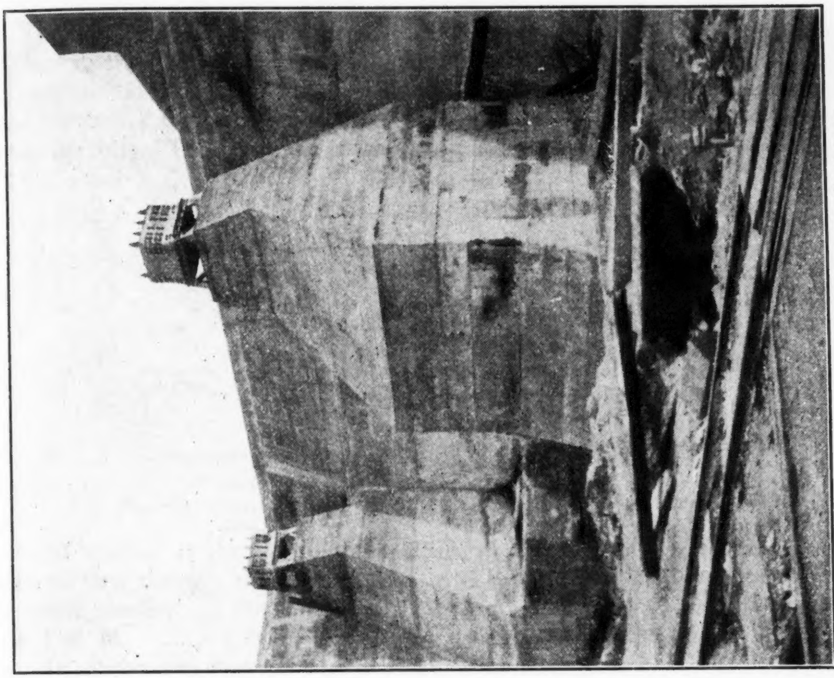

FIG. 11.—STEEL FOOTINGS ENCASED IN CONCRETE.

At those locations where the right-of-way width permitted, the accentricity of the footing was reversed, the toe pointing outward from the center of the structure. In this manner the shape of the footing was directly adapted to the support of the loads placed upon it, and its size and weight were reduced considerably. The resultant of combined horizontal arch thrust, and down

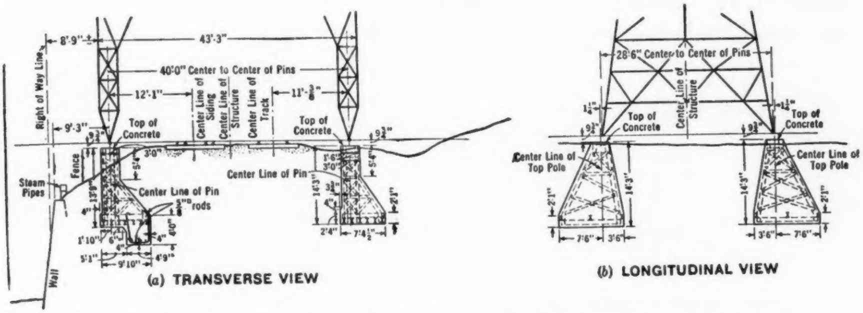

FIG. 12.—FOOTINGS REVERSED TO SUIT SPECIAL RIGHT-OF-WAY CONDITIONS.

thrust applied at the top of the footing, was inclined closer to the axis of the footing through the center of its base, and thereby an equalizing of the bearing pressure on the soil along the base was affected. This is illustrated in Fig. 12.

It will be seen that a generally similar shape of footing was used in many of these special cases, modified in each location to the extent required, in order to support the loads at that point and to conform to the outlines of the walls or abutments in which the steel frames were embedded. In all cases the steel framework was covered by a minimum of 3 in. of concrete.

At one section of the work, the Railroad Company had decided to raise its tracks about 20 ft. out of a cut, faced by retaining walls, to the level of the adjacent parallel street. The old railroad cut was filled in, with the tracks placed on the new fill. In order to support the shallow boundary wall above the surface, between the street and the tracks, piers in the line of the old retaining wall were carried down to spread bases below the railroad grade.

Footings constructed for the transmission structures in the usual manner would have been mainly in new fill. Hence, a steel column was devised to go down to the old subgrade under each pin of the superstructure on this side and bear directly upon a steel grid encased in concrete (see Fig. 13). Sufficient bearing support on firm soil was thus obtained, and the resistance to uplift was entirely satisfactory. Longitudinal thrust was taken at the tops of the columns through the reinforced concrete wall beam resting on the concrete piers of the railway. The earth, with the stability of the base itself, insured the footing against lateral movement due to transverse thrust. On the outside, the fairly packed subgrade of the street afforded resistance to the greater of these transverse pressures.

The columns of the railroad construction had to be enlarged to contain the steel framework, and they were reinforced with steel where necessary. All the load of the superstructure was carried through the steel substructure

to its base in firm soil at the old subgrade. This footing required the use of additional steel, but saved considerable concrete when compared with a concrete pier foundation under these conditions.

At a location similar to Fig. 4, high retaining walls with comparatively narrow bases were built by the Railroad Company. While they were stable under the railroad loads, these walls were insufficient in section to sustain the additional loads imposed by the tower structure above. It would have been

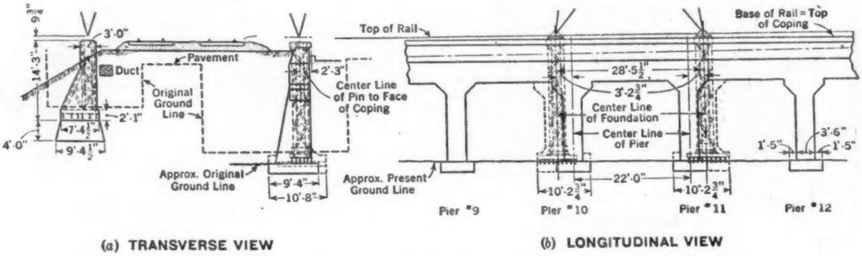

FIG. 13.—SPECIAL PIERS IN THE LINE OF AN OLD RETAINING WALL.

necessary to place considerable plain or reinforced concrete at or near the base of the walls and also to carry the steel of the footing framework to the base of the wall. The situation was made more acute due to the overhang of the steel superstructure at the top of the wall, in order to accommodate the standard width and maintain the required clearances.

In the case of all four footings, the vertical axis through the center line of the pins was close to the outside face of the wall (see Fig. 14). In view of these facts (and with the desire to save concrete even at the cost of a little more steel, so as to obtain the most economical substructure consistent with the best engineering), the footing arrangement shown in Fig. 14 was devised. The principles involved, consisted in carrying the bearing due to down thrust to a boxed truss, steel framework, placed in the wall in a longitudinal direction under each footing point. These frames extended from about 7 ft. on the outside of the pin to the center transverse axis of the structure, a space being left here between the frames under the two posts to allow independent action by the two footings. Thus, the bearing was distributed on concrete over a length of about 21 ft., and this influence, spreading downward at 45°, was dissipated in the concrete to cause only a small earth pressure at the bottom, in addition to the wall-bearing pressure. The overlapping of the 45°, downward pressure lines had little effect on this theory, except to reduce somewhat the effective area of spread at the bottom; and, generally, no effect at all when the two adjacent footings were subjected to opposite loadings. In uplift, the weight of concrete enclosed in these trusses in the wall proved more than adequate. The longitudinal thrust was likewise easily carried by the wall itself (see Fig. 15).

It remained necessary to provide for the large horizontal transverse thrust (applied at the pin connections), which caused a transverse shear, a spreading effect, and an overturning moment. By connecting the longitudinal

trusses with a transverse truss placed directly under the pins (partly shown in Fig. 16), the shear was taken care of, being transmitted through this truss and resisted partly by the longitudinal truss in the opposite footing, anchored in the wall and pulling against the fill. Overturning moment was resisted by the transverse truss through direct stress in the members causing, under different conditions, tension or compression in the top chord.

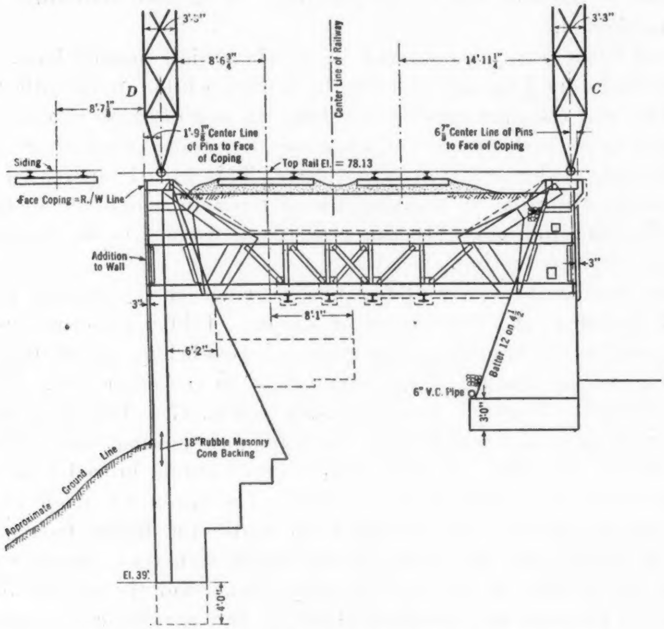

FIG. 14.—SPECIAL FOOTINGS FOR NARROW CLEARANCES.

The spreading effect was likewise resisted by the transverse truss. This truss was anchored by the weight of the concrete in which it was encased, by the weight of the earth above, and by its connections at the ends to the boxed bearing trusses in the walls. It was provided with triangular stiffener frames in the longitudinal direction, which rested on concrete seats. These stiffener frames were designed to take care of any longitudinal thrust, due to small amounts of tractive effect on the rails, transmitted to this depth, or other distortioning influences. The main transverse truss was designed to care for all tendencies of the walls to overturn, or fail outward at this point due to loads applied above its lower chord. It thus afforded an additional safety factor for the wall itself. This truss was also designed for the support of concentrated axle loads of 75 000 lb. each, on the track above, only one axle per track being applicable (longitudinally) to the truss at one time, due to its comparatively narrow section.

The bearing trusses were entirely within the normal wall and required no additional concrete. The transverse truss and stiffeners had to be encased

in concrete, but a comparatively small quantity of concrete was used for this purpose. In fact, this footing arrangement for the entire structure, while requiring more steel, was much more economical of concrete than the ordinary footing carried to the wall base. The bearing of the trusses on the wall concrete was accomplished through a grid of steel sections. The tendency of the steel frame to shear out locally, or for concrete to spall off at the face of the wall, due to the proximity of stressed structural steel, was duly investigated.

Several cases were encountered in which a high, narrow-base, retaining wall was to be used on one side of the tracks, while, on the other side, an earth slope was the final grade condition. In one of these places, the tower structure was located (because of clearance demands), close to an abutment for a through girder bridge over a street. This greatly increased the difficulties of the situation by limiting the distance in which the footing could be extended longitudinally. In this case it was decided to use a modification of the cross-bridge footing.

In the case of the two wall footings, bearing trusses, resting well up in the wall similar to the box trusses of the cross-bridge structure, were used. For support in the transverse direction, a cross-bridge would have been a useless expense on account of the earth slope on the other side. Therefore, a cantilever rib or bent was used as shown in Fig. 17. This bent was placed exactly at the pin, and projected in the rear of the normal wall section down to the base of the wall. At this point a small apron, braced longitudinally and transversely, was attached to the bent. The apron was made of concrete covering a steel frame, and extended out under the filling from the inner face of the wall. By this means, a horizontal transverse thrust at the pin increased the bearing on the wall to some extent; but the overturning effect was resisted through the cantilever bent by the weight of concrete in the apron, the direct vertical weight of earth in the fill above the apron (only the horizontal component of this earth acted on the wall), and a diagonal component of earth back of the apron and above the plane of rupture, extending upward from the inner edge of the apron. This structure was found to reduce the gross bearing on the wall materially by its great strength in resisting overturning.

Among the special foundation developments on this line was that caused by the use of short legs for the Type *R* bridge structure. This footing is illustrated in Fig. 3. The horizontal reactions of this structure in the transverse direction increased as the legs were shortened, until a point was reached where the foundations previously designed did not adequately resist this thrust. A footing was devised, therefore, with a large area of bearing surface in the vertical plane parallel to the right-of-way line, utilizing the lateral earth resistance. This type was found more economical than one which would resist the eccentricity of reaction by the addition of weight of concrete and increased bearing surface.

This type really consisted of three parts, as shown by Fig. 3, that is, a vertical "wall" section approximately 8 by 3 ft. at the top and 15 by 3 ft.

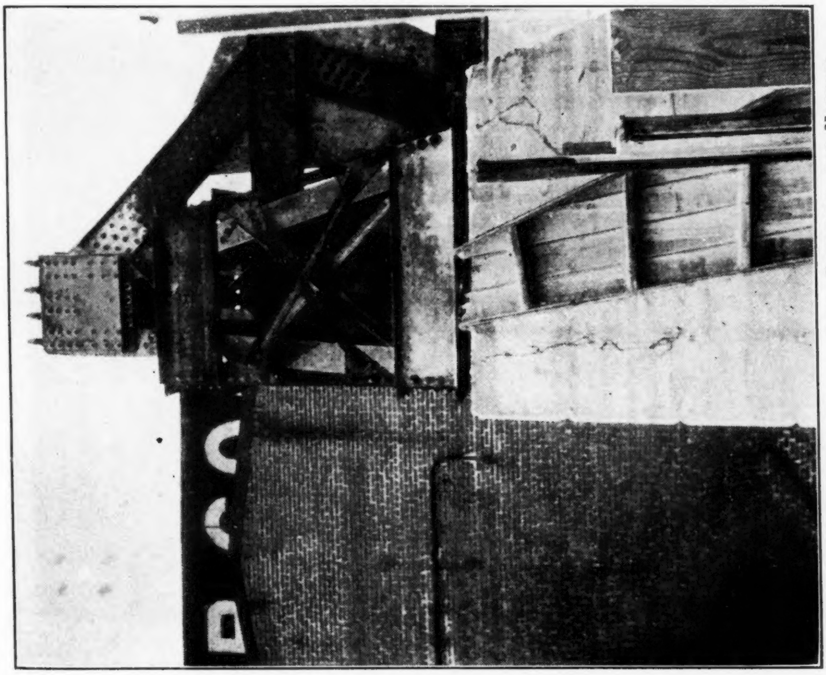

Fig. 15.—View of Special Pier Designed for Narrow Clearances.

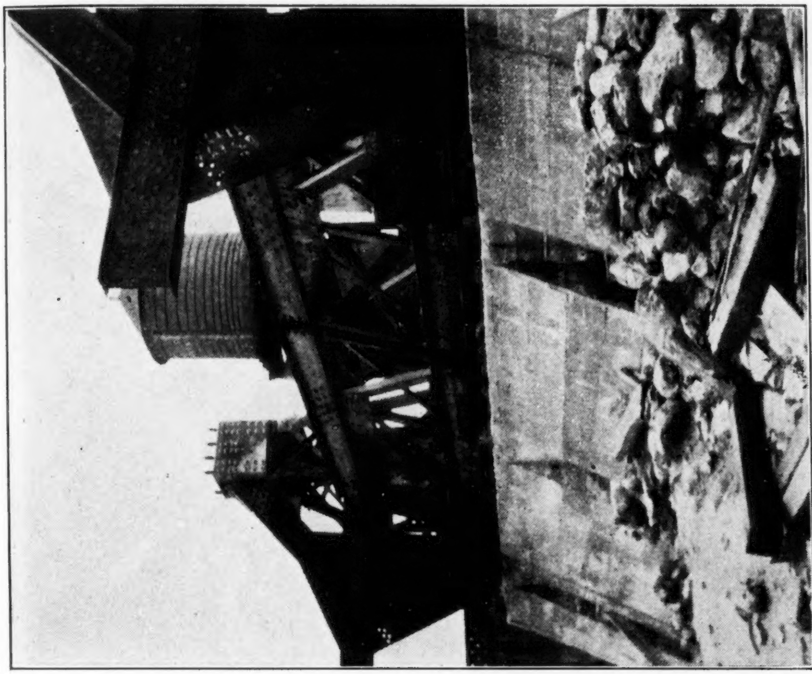

Fig. 16.—View Showing Connection of Longitudinal and Transverse Trusses, on Special Piers.

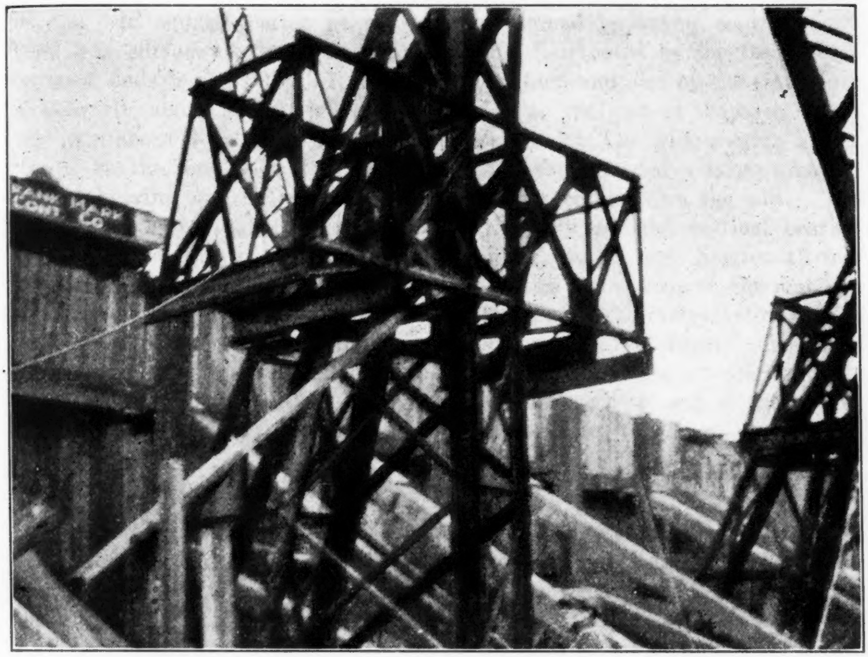

FIG. 17.—CANTILEVER RIB DESIGNED TO FURNISH SUPPORT IN A TRANSVERSE DIRECTION.

FIG. 18.—SPECIAL FALSEWORK USED IN ERECTION.

at the bottom. The concentrated load from the shoe of the bridge structure on the 3-ft. square section at the top, was spread by beams to the entire 8 by 3-ft. surface of the top. This load was distributed by the frame with battered ends to the 15 by 3-ft. area at the bottom, and also by the transverse sloping rib, about 13 ft. high and 3 ft. wide, to the mat at the base. This mat was about 6 by 15 ft. in area and 2 ft. thick. The wide section of the "wall" in the longitudinal direction near the top afforded a large area for lateral bearing on the soil near the transverse reaction at the pin. The large base, a total of 15 by 9 ft. in area, amply resisted vertical bearing pressure, and the rib combined the vertical "wall" and horizontal mat sections. In supporting the passive weight of the earth above it, the mat also resisted the overturning tendency due to the large transverse reaction. This proved to be an economical and simple solution of this difficulty. The steel frame was comparatively simple in design, little bending of angles being required. The usual grid of I-beams and channels was placed at the bottom of the framework, and the whole encased in concrete.

Construction

As stated previously, it has been the intention to confine the scope of this paper to design features. A brief description of the construction is interesting, however, in view of the close relation between it and the structural design.

The foundations were built in groups, primarily to facilitate a co-ordination of this construction work with that of the Railroad Company in its elimination of grade work and change of signal system. Accordingly, more than one-fourth of the footings were constructed, beginning about a year in advance of the superstructure work. These footings were at those locations where the steel framework was directly connected with the railroad walls or other structures. Approximately another one-fourth were constructed well in advance of the superstructure work, because they were in the path of or adjacent to conduits built for the new railroad signal system. The remainder were built as a part of the final operation in the construction of the line.

In several places in the foundation construction, unforeseen difficulties were encountered, which neither close inspections of the site, local experience, or borings had disclosed. The case of one of the cantilever types of wall footings is an example of this. The excavation of the base of the wall disclosed a soil condition entirely different from that for which the wall and footings were designed. Although at short distances on either side (less than 50 ft.) solid rock had been exposed, it was found necessary to place the wall and footing in this section on piles driven to refusal, the pile-heads being encased in a thick mat of reinforced concrete. Other cases of similar difficulty developed and were met by a corresponding change in the office design or by modification of the construction in the field.

In nearly all cases, construction operation was confined to the railroad right of way, thereby causing some restrictions as to methods of excavation, disposal of surplus material, and deliveries.

In many sections, access to the work was along the railroad tracks, or over ground inaccessible to trucks. It was possible to use trains on part of this job, but this type of construction auxiliary was distinctly hampered by the frequent passing of high-speed passenger trains.

Much of the excavation was in rock, although all types were experienced in the foundation construction. Concrete for the footings was partly mixed at the site or delivered from central mixing plants, along the job, and partly delivered ready-mixed from more distant sources.

The erection of the Type *M* Towers (Fig. 9) presented no peculiar difficulties; standard methods, a gin-pole, and hoist were all that was necessary. For the bridge structure, however, the problem was somewhat more involved. Usually, no large space was available for assembling the structure in sections; and even if there had been, the required locomotive cranes would have proved an uneconomical agent in this construction, due to lack of space for their operation and to their interruption of passenger-train service. Therefore, steel bridge falsework as illustrated in Fig. 18 was used. This framework was easily assembled, dismantled, and moved from site to site. The lower sections of the bridge post bents in the longitudinal view—that is, up to the first bend in the posts—were assembled, propped, and guyed in place. With the falsework in place over the tracks, light derricks were erected on it, and the lower transverse cross-bridge of the structure was quickly assembled. Then, using this as a falsework, the upper part of the structure was erected at will, the falsework having been moved in the meantime to another site. By this method, erection unit costs on this line were kept surprisingly low, quite comparable to unit costs of open-country, transmission-line construction.

With the exception of the great additional care required because of the location of this work over a railroad, the methods of carrying out the other phases of the work on this job were similar to the usual practice in ordinary transmission-line construction.

Conclusion

It has been the writer's intention to present a general picture of the design of this rather unusual type of transmission-line construction. The problem of power transmission has become increasingly acute with the development of widespread suburban residential districts around the manufacturing load centers. It becomes necessary to purchase right of way at high cost or to adopt costly design for a transmission line to bring power into a city from outside generating sources.

The line herein described presents one solution of this problem. The future may develop other and better solutions, all resulting in increased economies to power users and closer co-ordinating relations between utilities of all types.

The design of structures and foundations was done by the Engineering Department of Philadelphia Electric Company, and the construction of all superstructures and line erection and part of the footings by the United Engineers and Constructors, Inc. Other contractors built the remaining footings.

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PAPERS

STRESSES IN REINFORCED CONCRETE DUE TO VOLUME CHANGES

By C. P. Vetter,[1] Assoc. M. Am. Soc. C. E.

Synopsis

The purpose of this paper is to present a rational analysis of the stresses occurring in a continuous reinforced concrete structure which, due to variations in moisture content and in temperature, is subject to linear expansions or contractions.

Formulas and diagrams are given whereby the required reinforcement may be determined for any combination of the various agencies which produce volume changes.

Although cracks in structures of this kind cannot be entirely avoided it is possible, through suitable reinforcement, to make the individual cracks exceedingly small. It is shown, however, that the ratio of steel to concrete required for this purpose is relatively high and, in many cases, it will be considered prohibitive.

It is hoped, therefore, that one of the results of this investigation will be the increased use of definite contraction joints in all but very thin structures.

Notation.—Algebraic terms used in this paper are defined as they first appear and, in addition, are arranged alphabetically in Appendix I, for further reference.

Shrinkage Stresses

By "shrinkage" and "swelling" is understood the change in volume of concrete due to drying out and water-soaking, respectively, as distinct from the contraction and expansion due to changes in temperature. The shrinkage alone without temperature variations will first be considered.

Let it be assumed that, at some time during the initial shrinkage, cracks have formed a distance, L in. apart. These cracks are caused by the tension in the concrete having exceeded the ultimate tensile strength. The formation of the cracks will lower the concrete stresses temporarily, but as the shrink-

Note.—Written discussion on this paper will be closed in May, 1932, *Proceedings*.

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age proceeds, these stresses will again grow until they reach the tensile strength of the concrete, and new cracks will form between the earlier ones.

At any stage of this development there is a certain relation between the stresses in the block confined between two cracks, the shrinkage, and the distance, L. The first step is to determine this relation and then to use the equations established in this way on the ultimate stage of the development when (after the cracks, L in. apart, have formed) the concrete has again just reached the ultimate strength, and new cracks are just about to form.

Considering a unit, L in. long, between cracks formed at an earlier stage, it is readily seen that the concrete will attempt to shrink between the cracks, but that it is partly held back by the reinforcement steel. This will cause tension in the concrete, compression in the steel, and bond stresses between the steel and the concrete. These bond stresses are confined to the vicinity of the cracks. Beginning at a crack and proceeding toward the center of the block the bond will gradually produce increasing compression in the steel and tension in the concrete until a point is reached where the shortening per unit length of the steel equals the net shortening per unit length of the concrete, the latter being made up of the shrinkage less the elongation due to tension. Beyond this point there is, then, no relative movement of concrete and steel and, therefore, no bond stresses and no further changes in concrete and steel stresses. This picture of the stress distribution was first suggested by Professor Emil Mörsch for the somewhat similar case of a relatively short reinforced concrete block subject to shrinkage.[2] It has recently (and after the writing of this paper) been confirmed by tests conducted in England.[3]

It is furthermore realized that, for a continuous wall, the total length of the steel bars must remain unchanged. The steel, therefore, must be in tension for part of its length near the cracks, in order to offset the compression near the center of the blocks, the total shortening due to compression being equal to the total elongation due to tension. This will cause a certain amount of slippage between the concrete and the steel near the cracks.

The question arises as to whether this slippage affects the distribution of bond stresses. European investigators[4] have found that even a small slip decreases the bond materially. American tests, however, show[5] that the bond stress is not materially decreased by small movements. For the purpose of simplifying the mathematics, it is assumed that the bond stress is constant. The following conditions then prevail (see Fig. 1):

(1) At a point near the center of the unit, the shortening per unit length of steel and of concrete must be equal, or:

$$\frac{f'_s}{E_s} = z - \frac{f'_c}{E_c} \qquad (1)$$

in which, respectively, f'_s and f'_c are compression stress in steel and tension

[2] "Der Eisenbetonbau," Stuttgart, 1920, pp. 125–129; see, also, *Mitteilungen* über Forschungsarbeiten des Material-prüfungsanstalt, Stuttgart, Nos. 72–74.

[3] *Technical Paper No. 11,* "Studies in Reinforced Concrete II: Shrinkage," Dept. of Scientific and Industrial Research, Bldg. Research, H. M. Stationery Office, London, England; see, also, *Engineering* (Lond.), August 7, 1931, p. 181.

[4] *Zeitschrift* des Vereins Deutscher Ingenieure, 1911, p. 859.

[5] Concrete Engineers' Handbook, Hool and Johnson, New York, 1918, p. 267.

stress in concrete; E_s and E_c are the moduli of elasticity of steel and concrete; and z is the coefficient of shrinkage of concrete. Let $n = \frac{E_s}{E_c}$, then Equation (1) is,

$$f'_s = z\,E_s - n\,f'_c \qquad (2)$$

(2) The total bond stress must equal the total tension in the concrete, and also the total change in steel stress, or if x is the distance along the bar

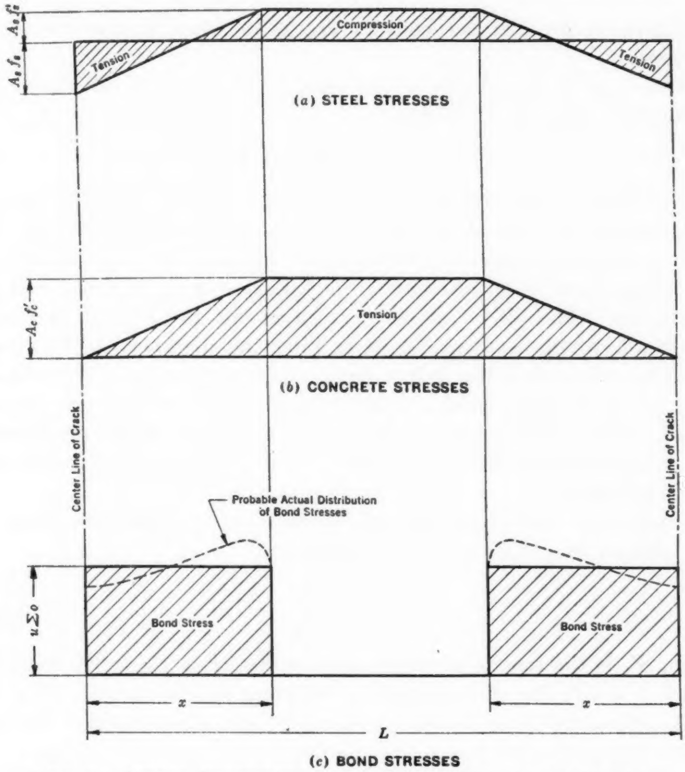

FIG. 1.—STRESS DISTRIBUTION BETWEEN CRACKS OF CONTINUOUS REINFORCED CONCRETE MEMBER SUBJECT TO SHRINKAGE

over which bond is effective, u = bond stress; Σo = circumference of the reinforcement bars; and A_c and A_s = concrete and steel areas, respectively:

$$u \times \Sigma o = A_c\,f'_c = A_s\,(f_s + f'_s) \qquad (3)$$

(3) The total shortening of the steel must be equal to its total elongation, or:

$$\frac{1}{E_s} \times \tfrac{1}{2} \times L \times f'_s = \frac{1}{E_s} \times \tfrac{1}{2} \times X \times (f'_s + f_s)$$

and,

$$L = \frac{x\,(f'_s + f_s)}{f'_s} \qquad (4)$$

Substituting from Equations (2) and (3):

$$L = \frac{A_c\, f'_c}{u\, \Sigma\, o} \times \frac{A_c}{A_s} \times f'_c \times \frac{1}{z\, E_s - n\, f'_c}$$

Let $p = \frac{A_s}{A_c}$; and $q = \frac{\Sigma\, o}{A_s}$; then,

$$L = \frac{(f'_c)^2}{n\, p^2\, q\, u\, (z\, E_c - f'_c)} \qquad \text{..................(5)}$$

which expresses the distance between cracks in terms of concrete tension and bond stress.

If there are to be no cracks, L must be infinite, or the denominator in Equation (5) must be zero; that is,

$$z\, E_c - f'_c = 0 \qquad \text{........................(6)}$$

With $z = 0.5 \times 10^{-3}$ and $E_c = 3 \times 10^6$, compression stress in the concrete, from Equation (6), would be, $f'_c = 1\,500$ lb. per sq. in., or several times the ultimate strength of concrete for tension. This confirms the well-known fact that reinforcement does not prevent shrinkage cracks, if the concrete is not free to move. The object of the reinforcement is to make the individual cracks small, which is identical to making the distance, L, between cracks small. From Equation (5) it is to be noted that L is inversely proportionate to p, q, and u; in other words, in order to get small cracks close together, the percentage of reinforcement should be great, as should the perimeter of the bars as compared with their area. The bars should be deformed so as to insure a high bond stress.

The maximum shrinkage that can be sustained by concrete, plain or reinforced, without any cracks, is determined by:

$$z = \frac{S'_c}{E_c} \qquad \text{..........................(7)}$$

in which, S'_c is the tensile strength of concrete. For $S'_c = 300$ lb. per sq. in., and $E_c = 3 \times 10^6$, the corresponding coefficient of shrinkage, z, for the concrete, is 0.10×10^{-3}, which is considerably less than normal shrinkage of concrete.

There is, however, another point which must be considered. The tensile stress in the steel, f_s, must not exceed the yield point, or the cracks will open.

From Equation (3):

$$f_s = \frac{f'_c}{p} - f'_s \qquad \text{......................(8)}$$

Substituting from Equation (2),

$$f_s = f'_c \left(\frac{1}{p} + n\right) - z\, E_s \qquad \text{...............(9)}$$

For $f_s = S_s$ (the elastic limit of the steel), and $f'_c = S'_c$:

$$p = \frac{S'_c}{S_s + z\, E_s - n\, S'_c} \qquad \text{.....................(10)}$$

The ratio, p, is inversely proportional to S_s and z; therefore, reinforcement steel with a high elastic limit, such as cold-twisted bars, should be used. The highest percentage of steel will be required when z is small, that is, in the beginning of the shrinkage period. It has been demonstrated that, for shrinkages less than the value determined by Equation (7), no cracks would form. This value, therefore, should be used for determining p; thus, in Table 1, Equation (11*a*) gives $p = 60\%$ for $S'_c = 300$ lb. per sq. in., and for $S_s = 50\,000$ lb. per sq. in.

The distance between cracks when the concrete stress has just reached the tensile strength, but before the new cracks form, is determined by Equation (11*e*) (Table 1). Thus, for $p = 0.60\%$, $u = 300$ lb. per sq. in., $q = 8$ ($\frac{1}{2}$-in. round bars), $E_s = 30 \times 10^6$, and $n = 10$, Equation (11*e*) shows that L varies from infinity to $L = 87$ in. when z varies from 0.10×10^{-3} to 0.50×10^{-3}.

These equations hold as long as $L \geqq 2x$. If the shrinkage should continue after the bond stress zone has reached the center of the unit, there would be no further changes in the stresses, but the cracks would increase in width due to continued slippage between concrete and steel.

The critical value of z may be determined by inserting values from Equation (3) and Equation (11*e*) (Table 1), in the equation, $L = 2\,x$, or,

$$\frac{(S'_c)^2}{n\,p^2 q\,u\,(z_1\,E_c - S'_c)} = \frac{2\ A_c\ S'_c}{u\,\Sigma\,o} \quad \ldots\ldots\ldots\ldots\ldots (15)$$

Solving for z, Equation (15) takes the form of Equation (11*d*) in Table 1.

If p is determined by Equation (11*a*) (Table 1), the critical value of z becomes,

$$z_1 = \frac{S'_s + 2\ n\ S'_c}{2\ E_s} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (16)$$

As before, let $S_s = 50\,000$; $S'_c = 300$; $n = 10$; and $E_s = 30 \times 10^6$; then the critical value, z_1, is found equal to 0.98×10^{-3}. Greater shrinkage than this will not occur with Portland cement concrete.

Temperature Stresses

The same reasoning which was used for the shrinkage stresses may now be applied to the temperature stresses (considered without accompanying shrinkage).

In this paper the term, "temperature drop," is used to denote the difference between the temperature at which the concrete has set and the temperature considered.

At first, before cracks appear, concrete and steel stresses are constant, that is,

$$\phi'_s = \epsilon\,t \times E_s \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (17)$$

and,

$$\phi'_c = \epsilon\,t \times E_c \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (18)$$

in which, the symbol, ϕ, represents a stress due to temperature drop as dis-

TABLE 1.—SPECIAL ARRANGEMENT OF IMPORTANT DESIGN FORMULAS

Item	Design condition	Shrinkage only, Equation (11)	Temperature drop only, Equation (12)	Indoor structures, shrinkage, and temperature drop, Equation (13)	Outdoor structures, swelling, and temperature drop, Equation (14)
(*a*)	Minimum reinforcement	$p = \frac{S'_c}{S_s}$	$p = \frac{S'_c}{S_s - n S'_c}$	$p = \frac{S'_c}{S_s - n S_c}$	$p = \frac{S'_c}{S_s - w E_s - n S'_c}$
(*b*)	Maximum temperature drop for minimum reinforcement..........................	....	$T = \frac{S_s + n S'_c}{2 \epsilon E_s}$	$T = \frac{S_s + n S'_c}{2 \epsilon E_s}$	$T = \frac{S_s + n S'_c + w E_s}{2 \epsilon E_s}$
(*c*)	Reinforcement for greater temperature drop....	....	$p = \frac{S'_c}{2 (S_s - T \epsilon E_s)}$	$p = \frac{S'_c}{2 (S_s - T \epsilon E_s)}$	$p = \frac{S'_c}{2 (S_s - T \epsilon E_s)}$
(*d*)	Critical volume change	$z_1 = \frac{S'_c}{E_c}\left(\frac{1}{2 pn} + 1\right)$	$\epsilon t_1 = \frac{S'_c}{E_c}\left(\frac{1}{2 pn} + 1\right)$	$\epsilon t_1 + z_1 = \frac{S'_c}{E_c}\left(\frac{1}{2 pn} + 1\right)$	$\epsilon t_1 - w_1 = \frac{S'_c}{E_c}\left(\frac{1}{2 pn} + 1\right)$
(*e*)	Distance between cracks....................	$L = \frac{(S'_c)^2}{n p^2 q u (z E_c - S'_c)}$	$L = \frac{(S'_c)^2}{n p^2 q u (\epsilon t E_c - S'_c)}$	$L = \frac{(S'_c)^2}{n p^2 q u [(\epsilon t + z) E_c - S'_c]}$	$L = \frac{(S'_c)^2}{n p^2 q u [(\epsilon t - w) E_c - S'_c]}$
(*f*)	Minimum distance between cracks	$L_{\text{min.}} = \frac{2 S'_c}{pqu}$	$L_{\text{min.}} = \frac{2 S'_c}{pqu}$	$L_{\text{min.}} = \frac{2 S'_c}{pqu}$	$L_{\text{min.}} = \frac{2 S'_c}{pqu}$

tinct from f which represented a stress due to shrinkage (the sub-symbols having the same meaning as before); ϵ represents the unit contraction per degree temperature drop; and t, the temperature drop, in degrees.

When the concrete stress exceeds the ultimate strength, the first cracks will form. Between the cracks the concrete attempts to contract, but is held back partly by the steel. There will be bond stresses between concrete and steel near the cracks and none near the center, where the stresses will be constant. The stress distribution is shown on Fig. 2. The distribution of bond

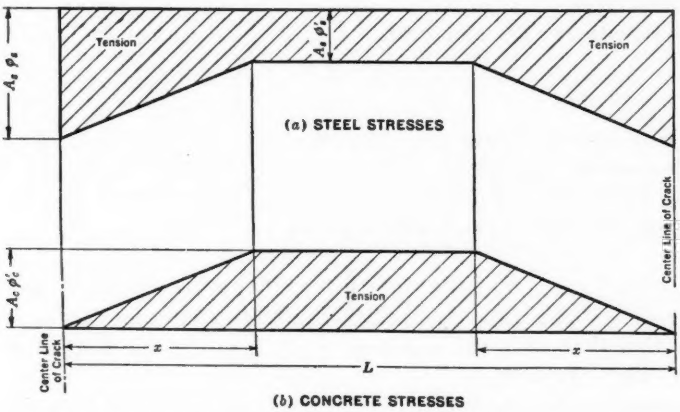

FIG. 2.—STRESS DISTRIBUTION BETWEEN CRACKS OF CONTINUOUS REINFORCED CONCRETE MEMBER SUBJECT TO TEMPERATURE DROP.

stress is the same as in Fig. 1(c), except that the length, x, should be represented by y.

The following equations may be written:

$$\frac{\phi'_s}{E_s} - \epsilon t = \frac{\phi'_c}{E_c} - \epsilon t \qquad (19)$$

which expresses the fact that near the middle of the unit there is no relative movement of concrete and steel;

$$y \times u \times \Sigma o = A_c \phi'_c = A_s (\phi_s - \phi'_s) \qquad (20)$$

which expresses the fact that the total bond stress is equal to the total concrete stress and to the total increment in steel stress; and,

$$\tfrac{1}{2} L \epsilon t E_s = \tfrac{1}{2} y (\phi_s + \phi'_s) + (\tfrac{1}{2} L - y) \phi'_s \qquad (21)$$

which expresses the fact that the shortening of the steel due to temperature drop is equal to the elongation due to tension.

Inserting appropriate values from Equations (19) and (20) in Equation (21):

$$L = \frac{(\phi'_c)^2}{n\, p^2 q u\, (E_c \epsilon t - \phi'_c)} \qquad (22)$$

or, for $\phi'_c = S'_c$, Equation (22) takes the form of Equation (12e) in Table 1.

Comparing Equations (5) and (22), it is seen that the distance between cracks depends on the unit contraction of the concrete and that it is independent of the manner in which such contraction has been brought about, whether by shrinkage or by temperature drop.

In the same way as for shrinkage, there will be no cracks if $E_c \epsilon t \leqq S'_c$; or, if $\epsilon t \leqq \frac{S'_c}{E_c}$. With the numerical values used previously, and with $\epsilon = 0.67 \times 10^{-5}$, $t = 15°$ Fahr. A temperature drop greater than 15° Fahr., therefore, will produce cracks.

From Equations (19) and (20),

$$\phi_s = \phi'_c \left(\frac{1}{p} + n\right) \qquad (23)$$

This discloses the interesting fact that the steel tension at the crack is independent of the magnitude of the temperature drop. At falling temperatures new cracks will form between the earlier ones, and always just before this happens, the steel tension will approach the value given by,

$$\phi_s = S'_c \left(\frac{1}{p} + n\right) \qquad (24)$$

From Equation (24) the required percentage of steel is determined by Equation (12*a*) (Table 1).

By comparing Equation (12*a*) with Equation (10) and Equation (11*a*) (Table 1), it is seen that, although, as a rule, the contraction due to shrinkage exceeds greatly that due to temperature drop, the reinforcement required by the latter is always the greater. With the numerical values used in the preceding examples, $p = 0.64\%$ by Equation (12*a*).

As before, Equations (19) to (24) hold good as long as $L \geqq 2\,y$. If the temperature drop continues after the bond stress zone has reached the center of the unit, no further cracks can form and the steel stress will increase.

The critical temperature drop is determined by inserting values of y and L from Equations (20) and (12*e*) (Table 1); thus,

$$\frac{(S'_c)^2}{n\,p^2\,qu\,(E_c \epsilon t_1 - S'_c)} = 2\,\frac{A_c\,S'_c}{u\,\Sigma o} \qquad (25)$$

or,

$$t_1 = \frac{S'_c\left(\frac{1}{2\,p\,n} + 1\right)}{\epsilon\,E_c} \qquad (26)$$

If p is determined by Equation (12*a*) (Table 1),

$$t_1 = \frac{S_s + n\,S'_c}{2\,\epsilon\,E_c} \qquad (27)$$

For the same numerical values as before, $t_1 = 132°$ Fahr. For temperature drops to the value given by Equation (27), minimum reinforcement may be used, as given by Equation (12*a*). If a still greater temperature drop may be expected, the reinforcement must be increased.

If the temperature drops an additional increment of t_2 degrees below the critical temperature given by Equation (27), so that the total temperature drop is $T = t_1 + t_2$, then,

$$t_2 \epsilon E_s = S_s - \phi_s \qquad (28)$$

Inserting ϕ_s from Equation (24),

$$t_2 \epsilon E_s = S_s - S'_c \left(\frac{1}{p} + n\right) \qquad (29)$$

or,

$$T = t_1 + t_2 = \frac{S_s - \dfrac{S'_c}{2p}}{\epsilon E_s} \qquad (30)$$

Solving for p, Equation (30) takes the form of Equation (12c) (Table 1). This is the value that should be used for temperature drops greater than the critical as determined by Equations (26) or (27).

Combined Shrinkage and Temperature Stresses

A reinforced concrete unit may be subject simultaneously to both shrinkage and temperature drop. The combined stresses may be distributed as shown on Fig. 3. It is there assumed that the temperature drop takes place first and is followed by shrinkage. The order might just as well have been reversed as the resultant total stresses would have been the same.

It is at once evident that the sum of Equations (2) and (19), and the sum of Equations (3) and (20) still hold good. Let, $F'_c = f'_c + \phi'_c$; and, $F_s = f_s + \phi_s$. Then, by adding Equations (3) and (20) and substituting from the sum of Equations (2) and (19):

$$F_s = F'_c \left(\frac{1}{p} + n\right) - z E_s \qquad (31)$$

For $F'_c = S'_c$, Equation (31) is identical to Equation (9) and the steel stress for combined shrinkage and temperature drop is seen to be less than that for temperature drop alone, as given by Equation (24).

It seems paradoxical that the addition of shrinkage to an existing temperature drop should reduce the steel tension. It must be remembered, however, that Equation (24) expresses an equilibrium in which the concrete between the cracks is just on the point of breaking. If shrinkage is added, the unit will break into shorter units in which the concrete stress is again equal to the tension strength, but the steel tension corresponding to this stress distribution is less than for temperature drop alone.

By substituting S_s for F_s, in Equation (31), and solving for p, an equation identical to Equation (10) is derived. As before, the highest value of p will be required when z is small; but the case for combined stress differs from shrinkage stress in that the minimum value for z in the former case is zero, while the smallest value in the latter case was expressed by $z = \dfrac{S'_c}{E_c}$.

The value of p to be used for design, therefore, is determined by Equation (12a) (Table 1). In other words, for combined shrinkage and temperature drop, the reinforcement is determined by the case of zero shrinkage. The

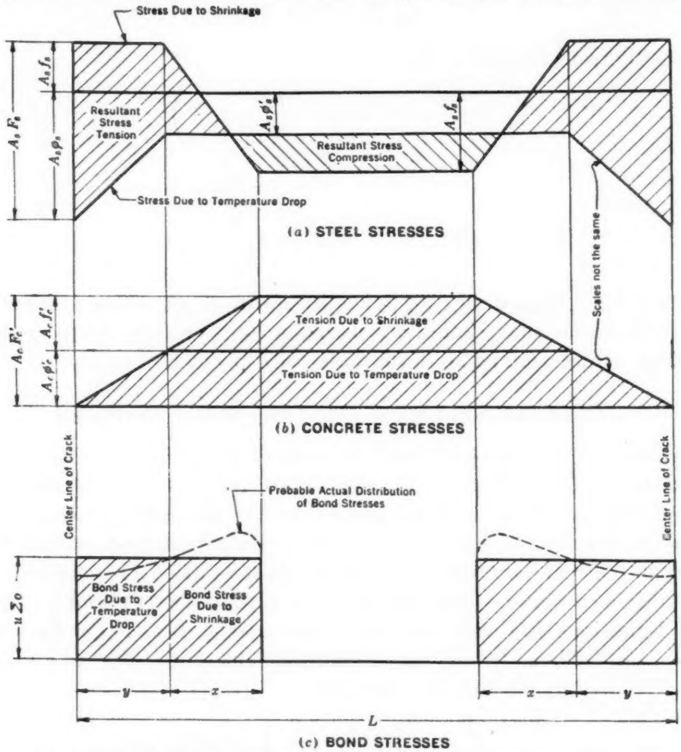

FIG. 3.—STRESS DISTRIBUTION BETWEEN CRACKS OF CONTINUOUS REINFORCED CONCRETE MEMBER SUBJECT TO SHRINKAGE AND TEMPERATURE DROP.

distance between cracks is found by expressing the condition that the total shortening of the steel is equal to the total elongation, or:

$$\frac{1}{2}\,y\,(f_s + \phi_s + f_s + \phi'_s) + \frac{1}{2}\,x\,(f_s + f'_s) = \frac{1}{2}\,E_s\,\epsilon\,t\,L + \left(\frac{L}{2} - y\right)(f'_s - \phi'_s)$$

By substituting x and y from Equations (3) and (20), and solving for L:

$$L = \frac{A_c\,(F'_c)^2}{u\,p\,\Sigma o + (E_s\,\epsilon\,t + z\,E_s - n\,F'_c)} \quad \ldots\ldots\ldots\ldots\ldots (32)$$

or, by substituting S'_c for F'_c, Equation (32) takes the form of Equation (13e) (Table 1).

By comparing Equation (13e) with Equations (11e) and (12e) (Table 1), it is seen that the distance between cracks is shorter for combined shrinkage and temperature drop, than is the case when any of the shortening agencies act alone. This explains the fact that the steel tension is less for combined shrinkage and temperature drop than for temperature drop alone.

The critical volume change is found from the condition: $2\ (x + y) = L$; thus,

$$z + \epsilon\, t_1 = \left(\frac{1}{2\,p\,n} - 1\right)\frac{S'_c}{E_c} \quad \text{......(33)}$$

which is similar to Equation (26) for temperature drop alone, and Equation (11*d*) (Table 1), for shrinkage alone.

If p is determined by Equation (12*a*) (Table 1):

$$z + \epsilon\, t_1 = \frac{S_s + n\,S'_c}{2\,E_s} \quad \text{......(34)}$$

If the temperature drops an additional increment of t_2 degrees below the critical temperature given by Equation (13*e*) (Table 1), so that the total temperature drop is $T = t_1 + t_2$,

$$t_2\,\epsilon\,E_s = S_s - F_s = S_s - S'_c\left(\frac{1}{p} + n\right) + z\,E_s \quad \text{......(35)}$$

which, with Equation (33), reduces to the form, Equation (30).

If p in Equation (30) is determined by Equation (12*a*) (Table 1), then, for combined temperature drop and shrinkage:

$$T = \frac{S_s + n\,S'_c}{2\,\epsilon\,E_s} \quad \text{......(36)}$$

For a temperature drop greater than that given by Equation (36), the reinforcement must be increased beyond the value given by Equation (12*a*) (Table 1). The value of p is found from Equation (12*c*) (Table 1), for combined stress due to temperature drop and shrinkage.

Combined Swelling and Temperature Stresses

In this paper shrinkage is understood to mean a decrease in length of the concrete unit due to drying. Concrete, however, may also be subject to swelling or an increase in length if it is submerged in water and is water-soaked. If the temperature remains constant, the concrete will be in uniform compression throughout its length and although this may cause difficulties in some structures, the subject lies outside the scope of this paper.

If, however, the temperature drops sufficiently to create a tension in the concrete beyond the tensile strength, cracks will form and a stress distribution will take place similar to that described for shrinkage and temperature drop. A typical stress distribution is shown in Fig. 4.

If swelling is treated as a negative shrinkage ($z < 0$), formulas may be established which are identical to Equations (9), (10), and (31). However, maximum reinforcement is no longer determined by $z = 0$, which condition leads to Equation (12*a*) (Table 1), but by the minimum value of z (maximum swelling).

If the unit swelling is given a positive value, w, then p is as given by Equation (14*a*) (Table 1) and Equation (13*e*) (Table 1) takes the form

of Equation (14*e*) (Table 1), indicating that cracks will form if $E_c\,(\epsilon\, t - w) > S'_c$, or $\epsilon\, t > \dfrac{S'_c}{E_c} + w$.

With $w = 0.1 \times 10^{-3}$, and other numerical values as before, $t > 30°$ Fahr. For temperature drops greater than that determined by the equation,

$$\epsilon\, t = \frac{S'_c}{E_c} + w \qquad \text{.......................(37)}$$

cracks will form even if the concrete has swelled.

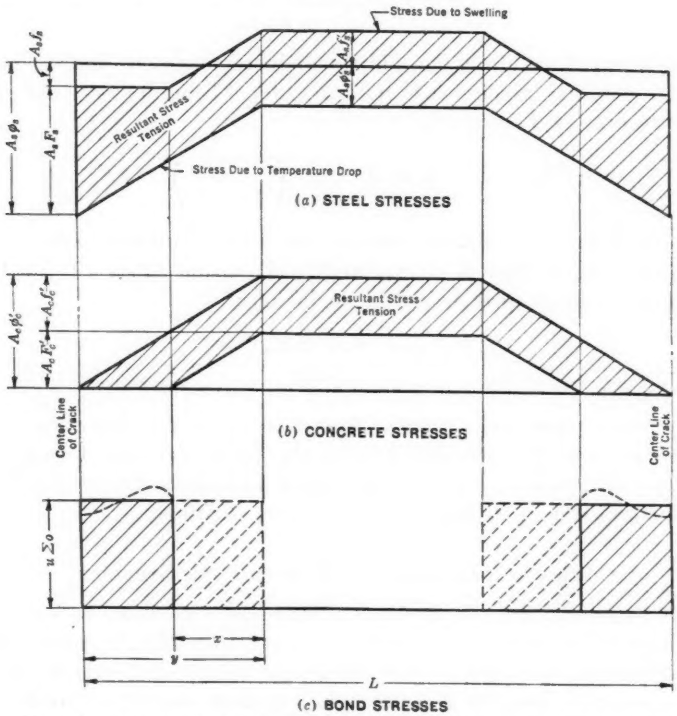

FIG. 4.—STRESS DISTRIBUTION BETWEEN CRACKS OF CONTINUOUS REINFORCED CONCRETE MEMBER SUBJECT TO SWELLING AND TEMPERATURE DROP.

Equation (33), giving the critical volume change, becomes:

$$\epsilon\, t - w = \left(\frac{1}{2\,pn} + 1\right)\frac{S'_c}{E_c} \qquad \text{..................(38)}$$

while Equation (30) remains unchanged. The reinforcement should be determined by Equation (14*a*) (Table 1), for temperature drops smaller than $T = \dfrac{S_s - \dfrac{1}{2p}\,S'_c}{\epsilon\, E_s}$, which may also be written as shown in Equation (14*b*) (Table 1).

For temperature drops greater than that given by Equation (14*b*) (Table 1), the reinforcement is determined by Equation (31).

The minimum distance between cracks for all cases discussed in this paper is given by Equation (11*f*) (Table 1).

Conclusions

The formulas which have been developed are applicable to such structures as concrete flumes and canal linings, retaining walls, road pavements, floors in warehouses and large buildings, etc., in which contraction joints have not been provided. It has been shown that a ratio of reinforcement of from 0.60 to 0.75% is required to prevent unsightly cracks or water leaks. These values are far in excess of what has hitherto been deemed necessary, but it is believed that practice has shown these earlier values to be inadequate. The general subject of volume changes in concrete due to water content has not been treated in this paper. Reference is made to exhaustive tests by Professor S. H. White which have been published from time to time over a period of twenty years by the American Society for Testing Materials.[6]

It has been shown that the greatest ratio of reinforcement is required for concrete subject, simultaneously, to swelling and temperature drop. Although the swelling would tend to counteract the effect of the temperature contraction, a temperature drop of about 30° Fahr. will be sufficient to create cracks in the concrete, and the steel stresses at these cracks are higher than for any other combination of volume changes. It has been shown, furthermore, that shrinkage alone and combined shrinkage and temperature drop are less dangerous than temperature drop alone without shrinkage.

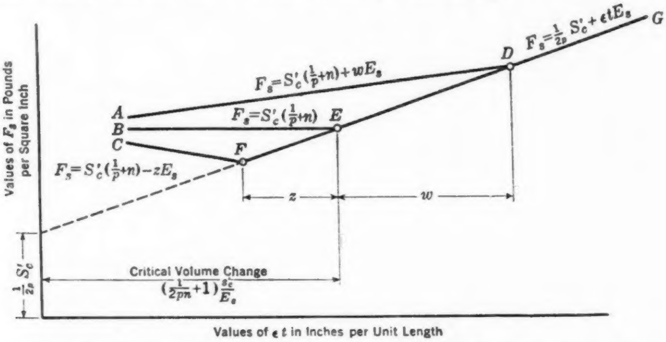

Fig. 5.—Variation of Steel Stress with Volume Change.

A volume change designated "critical" has been defined, which is dependent on the ratio of reinforcement and the strength characteristics of the concrete and the steel. For temperature drops in excess of the value corresponding to the critical volume change the temperature drop alone determines the reinforcement which is now independent of any volume change due to moisture.

The relationship between steel stresses and volume changes has been shown schematically in Fig. 5. It has there been assumed that the volume changes

[6] As, for example, *Proceedings*, A. S. T. M., Vol. 28, Pt. II, p. 398.

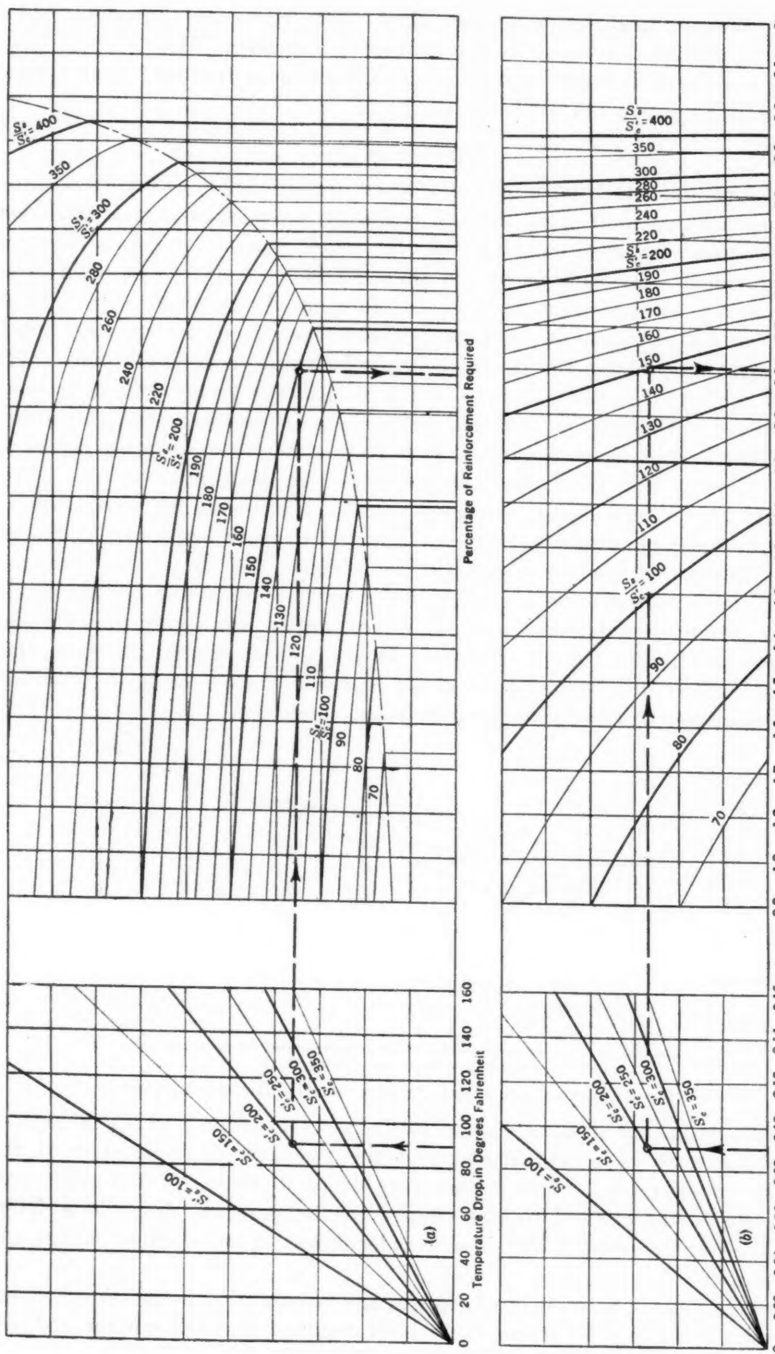

FIG. 6.—CURVES FOR USE IN DETERMINING PERCENTAGE OF REINFORCEMENT FOR ANY CONDITION OF VOLUME CHANGE.

due to moisture vary rectilinearly with the temperature drop. The diagram has no scale. Line *A D* represents swelling and temperature drop; Line *B E*, temperature drop alone, and Line *C F*, shrinkage and temperature drop. Points *D*, *E*, and *F* represent the temperature drops prevailing at the time the total volume change equals the critical value. Beyond these points the steel stresses vary rectilinearly with the temperature drop. It is interesting to notice that in this region the steel stress is greater by a constant quantity, $\frac{S'_c}{2p}$, than it would be in a continuous steel rod without any concrete.

For design purposes Lines *CFG* should not be used because a temperature drop without accompanying shrinkage is always possible. Lines *ADG* should be used for outdoor structures subject to swelling, and Lines *BEG*, for all other structures. A summary of the design formulas is given in Table 1. A diagram giving the required amount of reinforcement for any condition of volume change is shown in Fig. 6. In these curves, $n = 10$; $\varepsilon = 0.67 \times 10^{-5}$; and $E_s = 30 = 10^6$. Enter Fig. 6(*a*) for any given set of conditions and note the percentage obtained. If the structure is subject to swelling, check the first value by entering Fig. 6(*b*). Compare the values thus obtained and adopt the larger of the two.

APPENDIX I

NOTATION

The algebraic symbols introduced in this paper are defined as follows:

$f =$ unit stress due to shrinkage: f_s, tension in steel; f'_s, compression in steel; f_c, compression in concrete; f'_c, tension in concrete.

$n =$ ratio of moduli of elasticity, $\frac{E_s}{E_c}$.

$\Sigma o =$ sum of perimeters of bars.

$p =$ steel area ratio, $\frac{A_s}{A_c}$.

$q =$ ratio of perimeter of bars to area $\left(q = \frac{\Sigma o}{A_s} \text{ and } p = q = \frac{\Sigma o}{A_c}\right)$

$t =$ drop in temperature; $t_1 =$ temperature drop corresponding to critical volume change; $t_2 =$ additional drop.

$u =$ unit bond stress between concrete and steel.

$w =$ unit swelling caused by water-soaking.

x and $y =$ distances over which effective bond occurs.

$z =$ coefficient of shrinkage for concrete; $z_1 =$ critical value.

$A =$ area: A_s, of steel; A_c, of concrete.

$E =$ modulus of elasticity: E_s, of steel; E_c, of concrete.

$F =$ combined stress: $F'_c = f'_c + \phi'_c$; $F_s = f_s + \phi_s$.

$L =$ distance between shrinkage cracks.

$S =$ tensile strength: S'_c, of concrete; S_s, elastic limit of steel.

$T =$ total temperature drop.

$\epsilon =$ coefficient of thermal expansion of both steel and concrete.

$\phi =$ stress due to a drop in temperature: ϕ_s, steel stress at a crack; ϕ'_s, steel stress at the center between cracks; ϕ'_c, concrete stress at the center between cracks.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

REPORTS

WIND-BRACING IN STEEL BUILDINGS

SECOND PROGRESS REPORT OF SUB-COMMITTEE NO. 31, COMMITTEE ON STEEL, OF THE STRUCTURAL DIVISION [1]

Much valuable information and many helpful criticisms have been placed before the Sub-Committee by members of the Society and others since the presentation of the first progress report.[2] For these, whether favorable or unfavorable, the Sub-Committee is grateful. It would here reiterate its request for further co-operation on the part of the membership, since only by a pooling of the knowledge and experience possessed by engineers concerning wind-bracing can a satisfactory solution of the problem be reached in a reasonable time. Obviously, the formulation of any final report that might serve as the basis of a Manual of the Society will require long and painstaking investigation. In attempting this, the Sub-Committee proposes to proceed with deliberation, confining its attention at first to those phases of the problem that present the greatest opportunity for agreement and codification.

In this second progress report it is the intention to deal only with the following matters: (1) Comments on the discussion of the first progress report; (2) a method of analysis of shallow bracing systems; (3) arrangement of wind-bracing; (4) details of wind-bracing; and (5) new recommendations.

(1) Comments on the Discussion of First Progress Report

Prescribed Wind Force.—A wide range of opinion has been expressed by discussers concerning the wind force tentatively recommended by the Sub-Committee, namely, that for the first 500 ft. of height it be a pressure of 20 lb. per sq. ft., and that above this level it be increased at the rate of 2 lb. per sq. ft. for each 100 ft. of height. Some have definitely approved of the specified pressure, some are of the opinion that it is too high, and some think it is too low. D. C. Coyle, M. Am. Soc. C. E., would taper down the load from 20 lb. per sq. ft. at the 500-ft. level to 10 lb. per sq. ft. at the ground. Aubrey Weymouth, M. Am. Soc. C. E., believes that, on the basis of known

NOTE.—Written discussion on this report will be closed in May, 1932, *Proceedings*.

[1] Presented at the meeting of the Structural Division, New York, N. Y., January 21, 1932.

[2] *Civil Engineering*, March, 1931, p. 478.

performance, it is unnecessary to increase the pressure to 30 lb. per sq. ft. at the 1 000-ft. level. H. V. Spurr, M. Am. Soc. C. E., is of the opinion that the prescribed loading would produce excessively high horizontal shears in the lower stories of exceptionally high buildings. The late E. W. Stern, M. Am. Soc. C. E., and C. M. Goodrich, M. Am. Soc. C. E., and R. A. Philleo, Jun. Am. Soc. C. E., suggest varying the loading specification in accordance with the geographical location. S. P. Wing, M. Am. Soc. C. E., regards 30 lb. per sq. ft., at 1 000 ft. above the ground as "by no means generally conservative." Albert Smith, M. Am. Soc. C. E., considers the loading as much too low and suggests a stepped alternative, rising from 15 lb. per sq. ft. at the ground to 40 lb. per sq. ft. at the 500-ft. level and above.

It thus appears that the Sub-Committee actually took, as it had hoped to take, middle ground in the wind-force recommendation. There is, as yet, too much uncertainty respecting the behavior of the wind over the highly irregular skyline of a modern city to warrant any wide and sudden departure from existing practice. Speculation concerning the size, violence, or periodicity of gusts that swirl around and between buildings of various shapes and sizes is too insecure a basis on which to rest greatly increased demands with respect to wind, or, on the other hand, any substantial reduction of the force hitherto prescribed.

In the theory of wind action this much appears reasonable, namely, that the uniformly high velocity over a large frontage, that is associated with the freely moving gradient wind, is by the very existence of surface obstructions rendered impossible below the tops of the tallest structures present. Obstructions, although producing gusts, at the same time create relatively dead areas in front of some buildings and a shielding of others. These latter influences on the aggregate wind force on a building are believed to be very great, especially in the lower 500 ft. of height.

Experience lends support to this view. Many tall buildings with a satisfactory performance record have been designed for either an actually, or a virtually, smaller wind loading than has been suggested by the Sub-Committee. A force of 20 lb. per sq. ft., without restriction as to height, is now widely prescribed in building codes. Morever, large allowances have often been made for the uncertain and undependable factor of the wind resistance of walls and partitions. The fact is that building frames designed for less wind force than has been recommended by the Sub-Committee have, where proper details were provided, satisfactorily withstood winds of extremely high velocity, as at Miami, Fla., in 1926.

Mr. Philleo bases his doubt as to the adequacy of the wind-force recommendations of the Sub-Committee in part on the high estimated wind force required to cause the failure of certain structures in the St. Louis, Mo., tornado of 1896 and in the Central Illinois tornado of 1925. Against this should be cited the similarly calculated wind force of from 60 to 65 lb. per sq. ft. estimated by the Division's Committee on the Florida Hurricane of 1926 as necessary to cause distortion of certain structures at Miami.[3] In

[3] *Transactions*, Am. Soc. C. E., Vol. 95 (1931), p. 1119.

spite of this large computed force, buildings consistently designed for a wind load of 20 lb. per sq. ft. successfully resisted a wind with a probable velocity of more than 130 miles per hour. It would seem, therefore, that the Sub-Committee's wind-force prescription should be adequate for any part of the United States or Canada.

Any prescription of wind force cannot be fairly judged without fully taking into account the collateral stipulations respecting (*a*) stability; (*b*) permissible stresses in the material; (*c*) allowance, if any, to be made for the resistance contributed by walls and partitions; and (*d*) the limiting deflection. When the measures recommended to ensure stability and the prescribed stresses in material suggested in the first progress report and amplified herein are considered, together with the stipulation of Recommendation (4)[4] of that report that the frame be proportioned for 100% of the wind load, and with the further requirement of Recommendation (6)[3] that deflections and vibrations be kept within such limits as to render buildings comfortably habitable, it is evident that the recommendations of the Sub-Committee, when taken in their entirety, call for more provision for wind force than has often been made in important buildings. In any tall frame so proportioned as to ensure moderate deflection in service, the limitation of deformations through the use of necessarily conservative stresses in itself adds much to security.

Whenever experimental evidence becomes available upon which an amendment of the wind-force prescription may be based, the Sub-Committee will be ready to alter its recommendation in whichever direction it should be revised.

Limiting Stress in Members.—It was the intention of Recommendation (2)[3] of the first progress report, although not so stated in detail, that the method of securing stability should consist in requiring that the axial tension due to wind load in any column be not more than a certain fraction of the compression due to dead load in the column. It is believed, however, that, for the Sub-Committee's prescribed wind force, this limiting fraction should be not three-fourths, as suggested by Mr. Albert Smith, but two-thirds, and that the limit be operative unless the columns are securely anchored to the foundation by anchors proportioned at the permissible stress for members carrying wind load only.

After further consideration of its recommendation that in no member should the stresses due to the combined action of the prescribed wind force and all other loads exceed 75% of the elastic limit of the material, the Sub-Committee would amplify and replace it by the following:

For members or details subjected to wind stress only, except rivets and bolts, the permissible stress should be the same as that allowed for dead load or for dead load and live load.

For members subjected to stresses arising from the combined action of wind and other loads, and for rivets and bolts subjected to wind stress, the permissible stress, where the wind stress is $33\frac{1}{3}\%$ of the sum of the other stresses (or 25% of the total stress), should be $33\frac{1}{3}\%$ in excess of that allowed

[4] *Civil Engineering*, March, 1931, p. 483.

for dead load or for dead load and live load; for members in which the wind stress is of greater relative importance than this, the permissible stress should be progressively reduced until for a member subjected to wind stress only, the permissible stress is the same as that allowed for dead load, or for dead load and live load. This recommendation is made in order to ensure to parts carrying a large proportion of wind load a reserve of strength against increased wind load that is reasonably consistent with that realized in members in which the dead load and live load stresses predominate.

In conformity with this recommendation, the wind stress in members subjected to wind and other loads may be neglected if the wind stresses are less than $33\frac{1}{3}\%$ of the sum of the other stresses.

Members subjected to a combination of wind, dead load, and live load should, of course, have a section not less than the section required for dead load and live load alone at the normal working stresses.

Neglect of Walls and Partitions.—Some misconception of the intent of Recommendation (4) of the first progress report appears to have occurred. The Sub-Committee is appreciative of the undoubted dampening effect of walls and partitions on deflection and vibration, at least under the usual service conditions, but does not believe that any credit should be given them in strength calculations.

Limiting of Vibration.—The discussion thus far has not disclosed any definite quantitative facts upon which a building may be designed for a required performance with respect to deflection and vibration. Mr. Coyle is of the opinion that the sensation experienced by the occupants of a vibrating building depends upon some unknown function of amplitude and frequency. If this be true, the problem is then the determination of the degree to which sensation is affected by variations in each. While awaiting the solution, engineers may well direct attention to the means of modifying and controlling the constituents—amplitude and frequency. In this connection questions such as the following suggest themselves:

(*a*) What measures may be used in the design of the frame, or in the clothing of it, to minimize static deflection and amplitude of vibration?

(*b*) Is the deflection proportionate to the applied wind force, or does it increase more rapidly than the force?

(*c*) What is the relation between amplitude of vibration and static deflection?

(*d*) How does the apportionment of dead weight between the frame and the materials carried by it affect frequency?

(*e*) To what degree does a modern tall building in wind approximate a simple vibrating cantilever?

When answers to such questions are available, engineers will be better able to meet the problem of sensation.

(2) A Method of Analysis of Shallow Bracing Systems

In towers having a height of more than 500 ft. and a ratio of height to base of more than five, it is important that wind stresses be calculated with as great accuracy as is practicable, giving due consideration to the time

required for computation. It is also important that the resistance of the tower to deflection and vibration be ample to produce satisfactory behavior in service.

In building where deep wind-bracing, such as a triangulated or deep knee-braced system, is impracticable for architectural or other reasons, thus necessitating the adoption of joint details which cause the steel frame to act as a portal rather than as a triangulated truss, the method of analysis by distributing end moments, recently proposed[5] by Hardy Cross, M. Am. Soc. C. E., extended where necessary to take into account the effect of secondary moments, has been found, for buildings of moderate height, to produce excellent results from the standpoint of accuracy, and, at the same time, to be susceptible of use without undue expenditure of time and labor.

When this method is used, the designer, however, should review the results by a check-up of the vertical translation of the joints due to column shortening or lengthening under vertical wind load. The Cross method, as originally applied, assumes no such translation, but the same method may readily be used for the computation of the secondary bending moments resulting from vertical translation of the joints under wind load, and these moments combined with those originally determined will produce the final bending moments in the structure. Such a computation of secondary moments and their effects should be made, in general, in the case of high and narrow buildings.

For test purposes, an analysis by the Cross method was made of the 20-story building adopted by Professors Wilson and Maney in their classic paper developing the "slope-deflection" method of analysis.[6] This study kept in view the desirability of economizing time and labor, as it is well-known that the slope-deflection method, or any other of the so-called "exact" methods, is too laborious to be practicable without some form of abbreviation, and this is likely to result in an objectionable degree of inaccuracy, or at least of uncertainty as to the accuracy of the results.

The method of Professor Cross was applied to the entire 20-story building previously mentioned in accordance with the procedure outlined in the Appendix. Three cycles of operations were carried out for the lower half and two for the upper half. After this seemingly small amount of computation, the final moments at each column were estimated by a semi-graphical process also described in the Appendix.

It was seen that girder moments obtained upon the completion of the number of cycles mentioned in the foregoing were far from the correct values, but no further application of the Cross method was necessary on this account.

The column moments having been satisfactorily obtained, the girder moments were found from these by the condition, $\Sigma M = 0$, at each joint. For outside columns, this gave the girder moments directly. For an interior column, the sum of the two girder moments was obtained in this manner, and this sum was then divided between the girders in proportion to the last

[5] *Proceedings*, Am. Soc. C. E., May, 1930, Papers and Discussions, p. 919.

[6] *Bulletin No. 80*, Eng. Experiment Station, Univ. of Illinois, Urbana, Ill.

determined girder moments from the Cross analysis. The results closely approximated the true values. It was also seen that an application of the foregoing method of plotting successive moments was quite as satisfactory for girders as for columns, but the method of determining girder moments from previously ascertained column moments is preferable, as it ensures a perfect check of moments around each joint. However, the determination of the girder moments by plotting as described is recommended as furnishing an excellent all-around check of computed moments in both columns and girders.

The complete analysis of the building under consideration by the method just described resulted in column and girder moments which were within $8\frac{1}{2}\%$ of those determined by Professors Wilson and Maney, with the exception of two columns and one girder near the top, where the error was slightly more than 10%; but in each of these cases the actual value of the moment was neglibly small, being considerably less than 2 000 in-lb. Such errors as this are of no importance whatever when it is remembered that the wind assumption is a matter of much uncertainty and that the wind stress in any main member is usually not more than 25% of the total stress.

The actual time consumed in carrying out the computation was less than 2 days, and averaged less than 10 min. per member for the 80 members in the half tower, the structure being symmetrical about its vertical center plane. Naturally, in order to accomplish such a result in so short a time, it is necessary that the computer be thoroughly familiar with the process, so that full advantage may be taken of the almost mechanical nature of most of its operations.

The time of computation by this method is practically in proportion to the number of members, which is, of course, far from the case with the slope-deflection method, with its almost interminable solution of simultaneous equations. From the results obtained in the case in question, it is possible to estimate approximately the length of time required for computing wind moments in any proposed tower of similar construction.

The deflections of the Wilson and Maney bent for the lower four stories due to column moments were also computed at each floor and summed up to show the total deflection from that floor to the base, as indicated in Fig. 6 of the Appendix. This was done by a set of simple formulas which are also given in the Appendix. The work was performed in about a half day, which included a separate computation for each column. Actually, it is of course only necessary to compute the deflections for one column, since all columns must deflect equally in any one story; but it was considered advisable to perform the operation for each column, as this furnished a check from story to story, not only on the deflection calculation, but also on the column moments, as originally computed.

The total computed deflection may be compared with some assumed permissible deflection, and columns or girders, or both, increased in section where considered advisable in order to keep the deflection within proper limits.

The secondary bending moments in the tower resulting from the direct stresses in the columns were also computed by the method of Professor Cross, but, in the example studied, were found to be so small as to be relatively negligible. There being no deep wind-bracing, the direct stresses in the columns could produce no further deflection than that due to these small secondary moments. It was found, however, that the deflections computed as shown, were several times larger than those found by assuming the tower as a deflecting cantilever with direct column stresses producing deflection by reason of the distortion of a hypothetical triangulated web and chord system. This shows the desirability of designing such towers with a rigid deep system of wind-bracing where practicable.

While in the case of the 20-story Wilson and Maney bent with the center panel narrower than the outside ones, the moments and the panel shears did not require to be adjusted by reason of the effect of axial length changes in the columns, it is not represented that such adjustment is generally unnecessary. Systematic study of bents of various ratios of height to breadth, variously subdivided as to number and relative lengths of panels, should disclose the circumstances under which moment correction and shear redistribution become imperative.

The Appendix gives a summary of the processes used in the analysis, apart from shear redistribution, in order to facilitate its application. The method seems to combine a reasonable theoretical accuracy with a reasonable practicability as to required time and labor. It involves no higher mathematical operations, and is performed wholly by a simple and largely routine procedure, but it must be done with care and intelligent consideration. It naturally is not a task for the office boy, but should have the best attention of an experienced engineer. The importance of the subject demands nothing less.

For all the foregoing computations, a Thacher slide-rule was used with absolute satisfaction, and it is unhesitatingly recommended as fully meeting all requirements. A 20-in. Mannheim rule was adequate for moment computation, but somewhat less satisfactory for calculating deflections, where it was desirable to work with a greater number of significant figures.

(3) Arrangement of Wind-Bracing in Tier Buildings

In this report attention will be given only to wind-bracing arrangement in tier or multiple-story buildings, omitting such structures as armories, hangars, sheds, or other special types where the arrangement of bracing is clearly determined by the outlines of the structure itself.

In tier buildings engineers deal with a frame made up of vertical columns and horizontal floor members, which repeat at fairly regular intervals throughout the height of the building. The introduction of bracing is for the purpose of carrying the horizontal and vertical shears produced by the wind, thus preventing the collapse of the frame or its undue distortion. Sometimes the columns and floors make up very regular panels, but in other cases very irregular horizontal and vertical panels result from the requirements of the building layout.

Ordinary connections between beams, girders, and columns furnish a certain amount of rigidity to the frame. By increasing the capacity of these connections with the use of heavily riveted clip angles on the top and bottom of the horizontal member, or by using sections of **I**-beams in place of the angles, connections of considerable strength may be obtained. In this report these will be referred to as "knuckle" connections. Their action is to prevent distortion of the panels when horizontal force is applied. Analysis will show, however, that bending moments result in the columns and beams or girders, which in high or very narrow frames will mean considerable deflection in the building from "web distortion" alone.

The obvious advantage of knuckle connections is the ease of adapting them to conditions where diagonal bracing members would be objectionable in a building. In high or narrow frames, however, the inability to hold deflec-

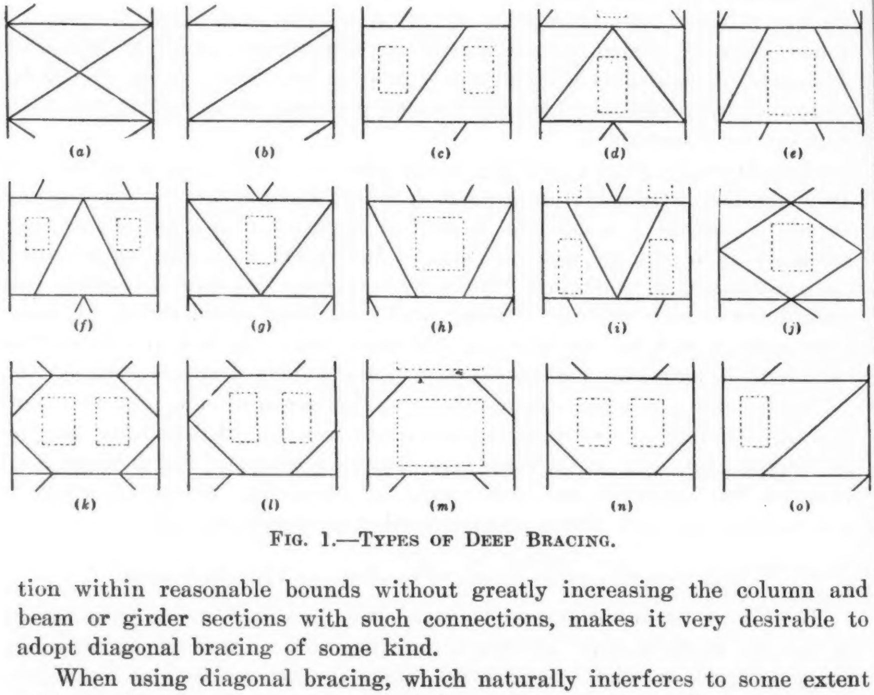

FIG. 1.—TYPES OF DEEP BRACING.

tion within reasonable bounds without greatly increasing the column and beam or girder sections with such connections, makes it very desirable to adopt diagonal bracing of some kind.

When using diagonal bracing, which naturally interferes to some extent with head-room, passageway, or window spaces, it is usually necessary to concentrate the resistance to horizontal force in certain panels or bents of the building where their use will not seriously affect the plan. The outside walls usually offer such an opportunity. Other chances occur between elevators and around permanent shaft or service areas. Frequently, particularly in set-back buildings, lines can not run continuously from top to bottom of the building. This does not create serious difficulty, however, as provision for horizontal transfer of shears can usually be made in the floor construction at the offset level. In the arrangement of diagonal bracing the designer can

often exercise considerable ingenuity so as to avoid necessary passageways and openings. Common arrangements are indicated in the diagrams of Fig. 1.

If each bent which is to carry wind is pictured as a vertical truss, with the columns forming chords and the floor members the web posts, the most natural and effective way to complete the web system is to add diagonals crossing the panels. The most effective and economical bracing will approach this simple solution.

Obviously, then, it is desirable, wherever possible, to provide a simple triangulation of the rectangle formed by two consecutive columns and floors, as in Fig. 1 (*a*), (*b*), (*d*), and (*g*). One objection to the type involving either a single diagonal, as in Fig. 1 (*b*), or double diagonals, as in Fig. 1 (*a*), is that, due to column shortening under vertical load, the diagonals receive load for which they were not intended. This can be largely overcome by detailing the diagonals short and pre-stressing them in tension as they are erected.

Modifications of the single diagonal idea are indicated in Fig. 1 (*c*) and Fig. 1 (*o*) and of the double diagonal idea in Fig. 1 (*d*), (*e*), (*f*), (*g*), (*h*), (*i*), and (*j*). In these types the effect of column shortening is less serious than in the types of Fig. 1 (*a*) and Fig. (*b*).

The *K*-type, indicated in Fig. 1 (*d*) and Fig. 1 (*g*), is a very adaptable and useful type, providing great rigidity against lateral drift in the story while at the same time involving relatively small participation in axial column loads. When conditions require, the *K*-type can be opened at the points, as in Fig. 1 (*e*) and Fig. 1 (*h*), or the arms may be drawn in along the girder from the column intersections as in Fig. 1 (*f*) and Fig. 1 (*i*), to facilitate arrangements for windows and doors. Where the diagonals of the *K*-type intersect the girder at a substantial distance from its ends, the girder may be profitably designed as a continuous beam over the supports afforded by the bracing. The resulting saving in girder weight may offset in great measure the cost of the bracing in the panel. Selection of the exact type is usually determined by the necessary avoidance of openings or other architectural requirements.

It is often necessary to use one type in one panel of a bent and another type in adjoining panels. This offers no great difficulty, but analysis must be based on equal web deflections in such panels so that the bent may work as a unit, as assumed. It will usually be found difficult to combine such extremes as a knuckle-braced panel with a full diagonal-braced panel in the same bent or line. Without great waste of material the knuckle panel can hardly be made stiff enough to take any considerable load in comparison with the fully braced panel.

Deep riveted gusset-plate connections have been used in many wind designs, and still are used. To obtain a rigidity equal to that of a diagonally braced panel, however, usually requires more material and many more rivets in such a type, with consequent loss of economy.

(4) Details of Wind-Bracing

In arranging effective details for wind-bracing it should be the effort to permit as little "give" as possible in them, bearing in mind that rigidity is sought in the frame, and that "give" in the connections will contribute to the deflection. In a simple case of a connection, as illustrated in Fig. 2 (*a*), it is desirable to keep the rivets attaching the clip angle to the column near the root of the angle, in order to avoid bending in the angle. Analysis will show that one line alone, and that kept near the root of the angle, is all that can be made effective. For the same reason the rivets in the column should be kept in single gauges near the column web.

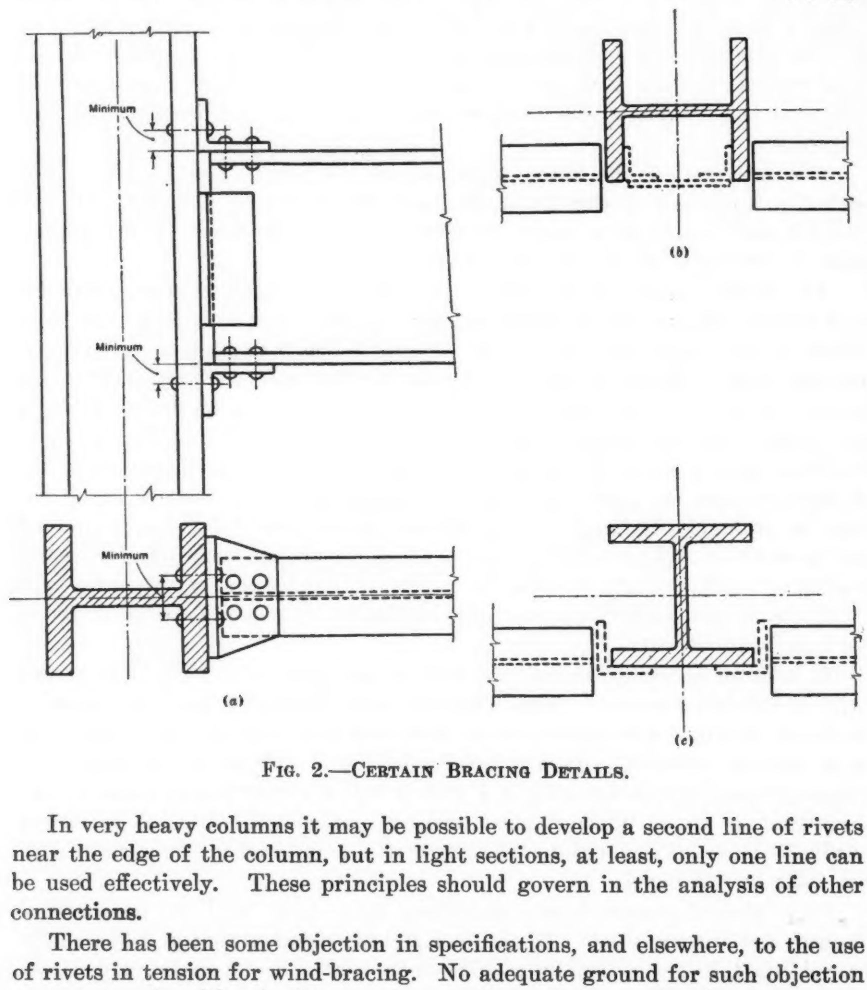

Fig. 2.—Certain Bracing Details.

In very heavy columns it may be possible to develop a second line of rivets near the edge of the column, but in light sections, at least, only one line can be used effectively. These principles should govern in the analysis of other connections.

There has been some objection in specifications, and elsewhere, to the use of rivets in tension for wind-bracing. No adequate ground for such objection appears to exist either in theory or in practice, and thousands of such arrangements have worked satisfactorily. Tests have shown that rivets in tension

possess a strength at least as great as the bars from which they were made, and that extension of the rivet shafts does not become appreciable until the yield point of the material is reached.

Welding will no doubt find its place in wind-bracing details in the near future, particularly for field connections. Compactness will be desirable for such connections, however, just as for riveted connections, to reduce the "give" and consequent deflection of the frame. It is possible that increased rigidity may be secured by the use of welding, which readily lends itself to to extremely compact joints.

In placing wind-bracing in outside walls it is customary to find the line of bracing several inches off the column center, as in Fig. 2 (*b*) and Fig. 2 (*c*). The vertical load transmitted from the bracing to the column should be computed like any other eccentric vertical load, in such a case. The horizontal load transmitted is usually small at any floor, and, therefore, can be neglected. If the bracing is arranged so as to put bending in the columns, as illustrated in the right-hand column of Fig. 1 (*l*), some difficulty may arise, particularly if the attachment is made as in Fig. 2 (*b*). With the detail indicated in Fig. 2 (*c*), a great deal of bending can be met by simply adding angles to the column, thus making a rigid girder section of the outside flange. The detail of Fig. 2 (*b*) is not suitable for use where much column bending occurs.

(5) Conclusions and Recommendations

For convenience, the further views herein advanced and the new recommendations made, may be summarized as follows:

(1) No experimental evidence concerning the behavior of the wind has come before the Sub-Committee during the past year that would warrant a change in the wind force prescribed in the first progress report. Having regard to the collateral stipulations respecting stability, permissible stresses, participation of walls and partitions, and limiting deflection, it is believed that, in the light of present knowledge, the prescription is reasonable and adequate.

(2) Stability under the recommended wind force should be ensured by requiring that the axial wind tension in any column be not more than two-thirds of the dead load compression, unless the column is securely anchored to the foundation.

(3) For members or details subjected to wind stress only, except rivets and bolts, the permissible stress should be the same as that allowed for dead load or for dead load and live load.

For members subjected to stresses arising from the combined action of wind and other loads, and for rivets and bolts subjected to wind stress, the permissible stress, where the wind stress is $33\frac{1}{3}\%$ of the sum of the other stresses, should be $33\frac{1}{3}\%$ in excess of that allowed for dead load or for dead load and live load; for members in which the wind stress is of greater relative importance than this, the permissible stress should be reduced progressively

until, for a member subjected to wind stress only, the permissible stress is the same as that allowed for dead load, or for dead load and live load.

Wind stress in members subjected to wind and other loads may be neglected if the wind stresses are less than $33\frac{1}{3}\%$ of the sum of the other stresses.

(4) Wind bents of moderate height in which a shallow bracing system is used may be analyzed for bending moments in the members by the Cross method of moment distribution with excellent results, both from the standpoint of accuracy and from that of saving of labor. In general, particularly for tall, narrow frames, the effect of length changes in the columns should be investigated.

(5) Deflections of frames with shallow bracing may be readily determined from previously found column moments by a method herein outlined.

(6) The most effective and economical bracing will approach a simple triangulation. In high, narrow frames the inability to hold deflection within reasonable limits with shallow systems of bracing without greatly increasing column and girder sections makes it highly desirable to adopt some form of diagonal bracing. It is often possible by using the K-form of bracing to make the desired provision for passageways and windows and, at the same time, to effect such a saving of weight in the girders as to offset in large measure the cost of the bracing in the panels where it is used.

(7) Proper use of one type of bracing in one panel of a bent and another type in adjoining panels must be based on equal web deflections of the panels. It is, in general, impracticable to combine types of widely different essential rigidities in the same story of a bent.

(8) In the connection of bracing members to columns and girders, attention should be given to the placing of rivets in such locations as to minimize the deformation of connecting details and heighten the effectiveness of the rivet group.

(9) Rivets acting at a reasonable stress in tension are entirely satisfactory, both with respect to security and rigidity.

(10) Provision for bending in columns due to the action of knee-braces should be made, where necessary, either by reinforcement or by diaphragms.

Respectfully submitted,

Clyde T. Morris,
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Francis P. Witmer,
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Chairman.

December 21, 1931.

APPENDIX

Recommended Procedure in Moment Calculations for Shallow Bracing Systems.—The initial procedure recommended for the computation of wind moments in a bent in which a shallow bracing system is used, is that outlined later. The various steps may be readily followed by reference to Fig. 3,

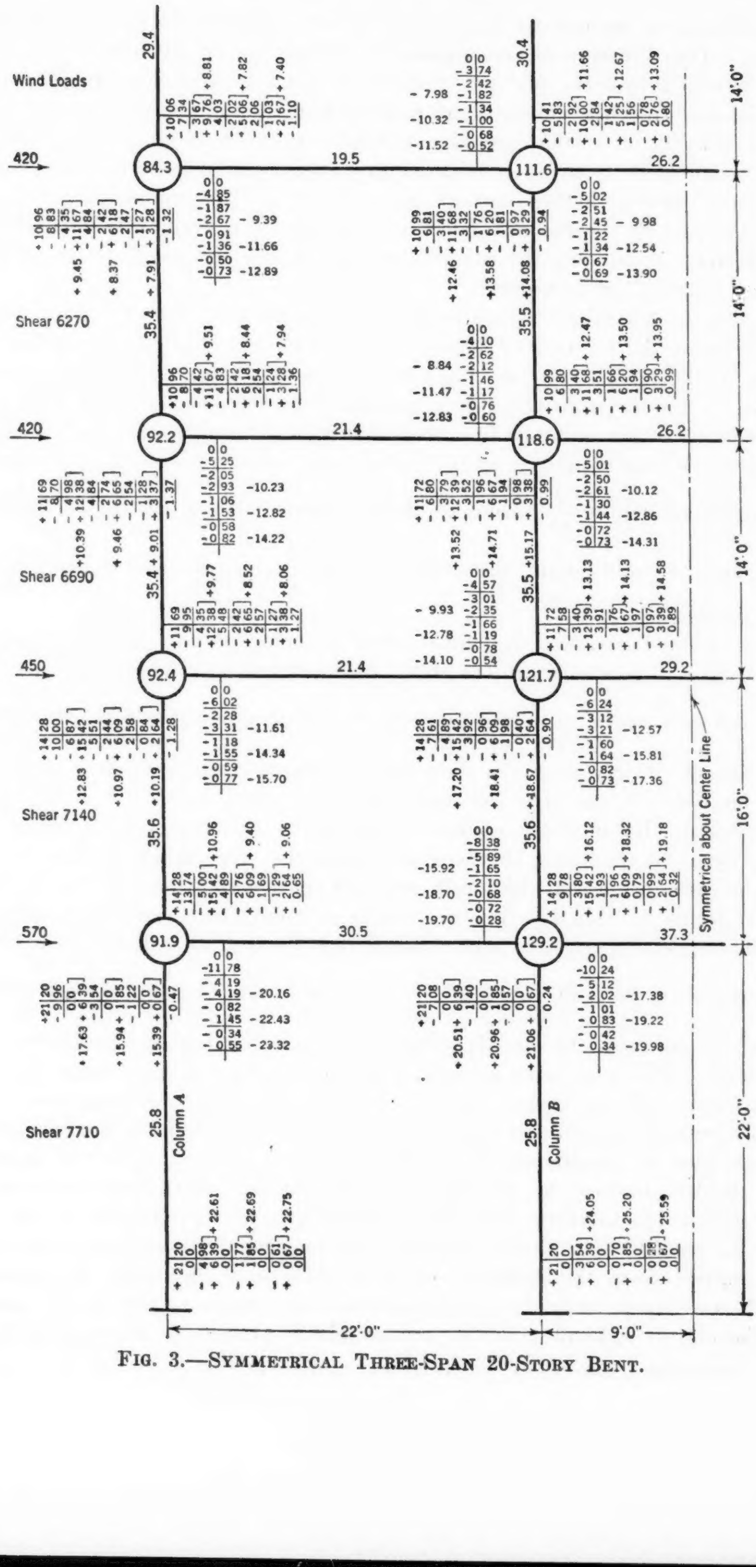

Fig. 3.—Symmetrical Three-Span 20-Story Bent.

pertaining to the bottom four stories of the symmetrical Wilson and Maney bent. This illustration accompanied a discussion[7] on Professor Cross' paper by Clyde T. Morris, M. Am. Soc. C. E., and is here reproduced for convenience. In it the moments are expressed in thousands of foot-pounds. The sign convention is that any moment in a member which tends to rotate a joint clockwise is to be considered as positive and the reverse as negative. Moments shown in the diagram are internal resisting moments.

Analysis by the recommended method may be conveniently carried out in four steps, as follows, the moment of inertia and the length of any member being I and L, respectively:

Step 1.—Calculate the moments in the columns due to the lateral forces, considering the joints fixed against rotation, but free to deflect laterally. The sum of the moments at the top and bottom of all the columns of a story is equal to the shear in the story, multiplied by the story height, and, as the deflections of the columns in a story due to the lateral forces are equal, neglecting the length changes in the girders, the column moments and shears are proportional to the $\frac{I}{L^2}$ values of the columns. (When all the columns of a story are of equal height, the values of I, or $\frac{I}{L}$, may be used in proportioning the moments.)

Step 2.—Distribute the unbalanced moments at the joints, considering them free to rotate, all intersecting members taking moments in proportion to their values of $\frac{I}{L}$. These moments have opposite signs to those in Step 1.

Step 3.—Carry over the distributed moments, using a carry-over factor of one-half and retaining the same signs as in Step 2.

Step 4.—Balance the column moments in each story by making their sum equal to the shear in the story times the story height. This is based on the assumption that the joints are fixed against rotation, but free to deflect laterally, as in Step 1. The correcting moments to be added or subtracted in this operation must be apportioned, therefore, to all columns in the story in proportion to their values of $\frac{I}{L}$ (if all columns of the story are of equal height), and must be equally divided between the top and bottom of each column. This completes a cycle, and the designer is now ready to repeat Steps 2, 3, and 4 as many times as the desired accuracy may require.

In general, it will be necessary, particularly with high narrow frames, to consider the modifying effect of the vertical translation of the joints on the results obtained by the foregoing procedure. Fixed-end moments for any girder due to vertical displacement of one end with respect to the other may be readily ascertained, the joints balanced, and the revised panel shears determined from the moments existing after final balancing is completed.

A convenient method of approximating the limiting value of the moment at the end of a member is the semi-graphical process to which allusion has

[7] *Proceedings,* Am. Soc. C. E., February, 1931, p. 321.

been made in the body of this report. The values of the moments for Fig. 3 for each end of each column at the beginning of the analysis and at the end of each cycle of operations were plotted horizontally with uniform vertical departures, giving, when joined, the curves indicated in Fig. 4. These curves were seen to be convergent, so that, by drawing in successive chords of equal departures, making the angle of each with the preceding chord

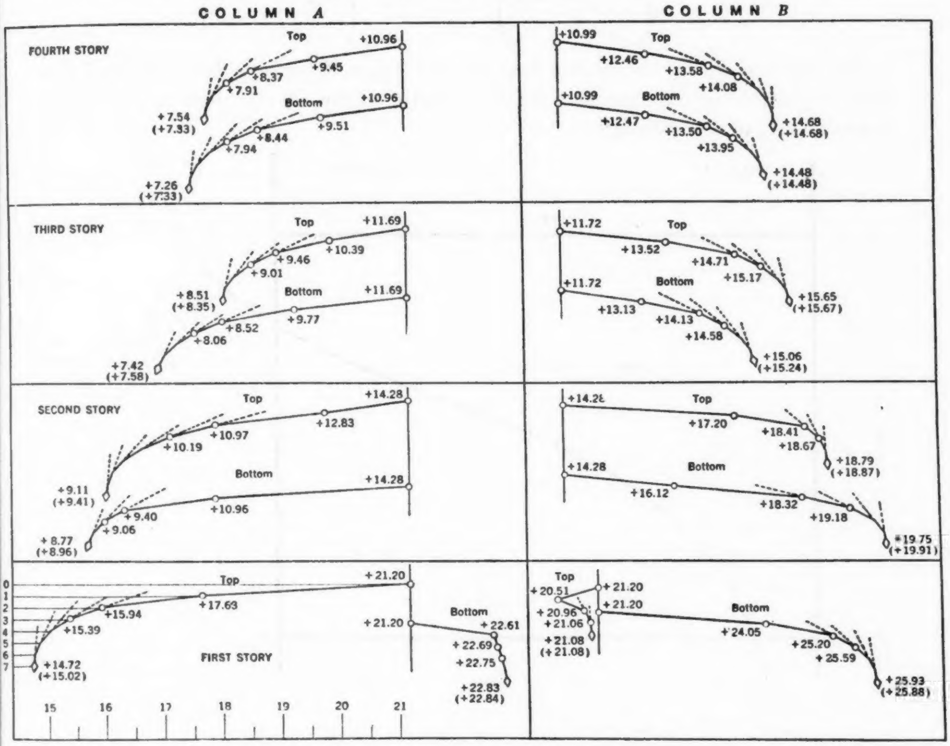

Fig. 4.—Semi-Graphical Method of Approximating Final Column Moments.

slightly greater than the last angle, a maximum point or a minimum point was quickly obtained, which was found to be well within satisfactory limits of accuracy of, say, 10 per cent. The chords were drawn very quickly, the preceding angle being readily carried in the eye while drawing each successive chord. The maximum or minimum value of the moment is recorded by a diamond and, also, in parentheses, the moment as found by Professors Wilson and Maney by the slope-deflection method. The differences never exceed about 3 per cent. For the cases shown, four points are available for each curve. For the upper half of the 20-story bent only three points are available, but these proved sufficient for equally accurate results.

Procedure for Computation of Story Deflection Due to Column Moments.—Let Fig. 5 represent any story between two columns. Let the curved line represent the deflected elastic curve of Column A in this story, the moment of inertia of which is I.

Let M_B and M_T be the moments in this column at the bottom and top of the story, respectively.

Let l_B and l_T be the distances from the top and bottom of the story, respectively, to the point of contraflexure of the column. Then,

$$l_B = \frac{M_B}{\left(\frac{M_B + M_T}{L}\right)}; \qquad l_T = \frac{M_T}{\left(\frac{M_B + M_T}{L}\right)}$$

Let θ_B and θ_T be the angles between the original vertical direction and the final direction of the tangent to the elastic curve at the bottom and top, respectively, of the column.

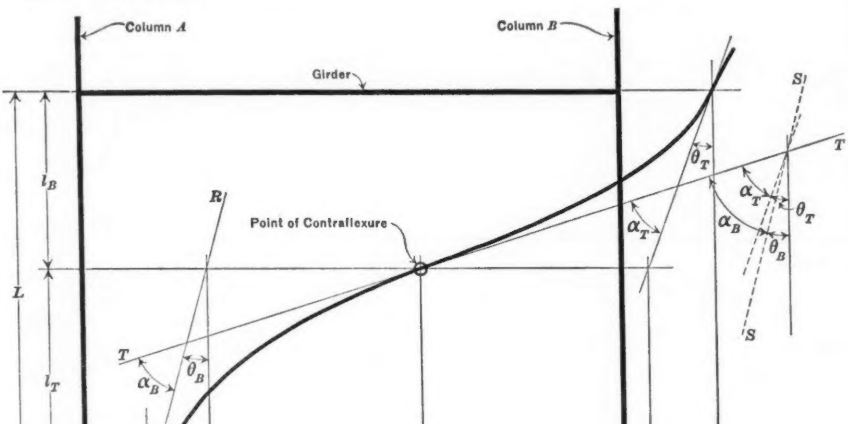

FIG. 5.—DEFLECTION IN STORY DUE TO COLUMN MOVEMENT.

Let T-T be the tangent at the point of contraflexure. Then,

$$\alpha_B = \frac{M_B\, l_B}{2\, EI}; \qquad \alpha_T = \frac{M_T\, l_T}{2\, EI}$$

the tangents at top and bottom being considered as deflecting cantilevers fixed at these points.

Drawing S-S parallel to R-R, it is seen that,

$$\theta_T = \theta_B + \alpha_B - \alpha_T$$

or,

$$\theta_T = \theta_B + \frac{M_B\, l_B - M_T\, l_T}{2\, EI} \quad \ldots\ldots\ldots\ldots\ldots (1)$$

For the first story, $\theta_B = 0$. For any other story, $\theta_B =$ the value of θ_T for the story below.

The deflection for the story is,

$$\delta = d_1 + d_2 + d_3 + d_4$$

in which, in consideration of the extremely small values of the deflection angles, θ_B and θ_T,

$$d_1 = \theta_B\, l_B$$

$$d_2 = \frac{M_B\, l_B^2}{3\, EI}$$

$$d_3 = \frac{M_T\, l_T^2}{3\, EI}$$

$$d_4 = \theta_T\, l_T$$

Hence,

$$\delta = \theta_B\, l_B + \theta_T\, l_T + \frac{M_B\, l_B^2 + M_T\, l_T^2}{3\, EI} \quad \ldots\ldots\ldots\ldots\ldots\ldots (2)$$

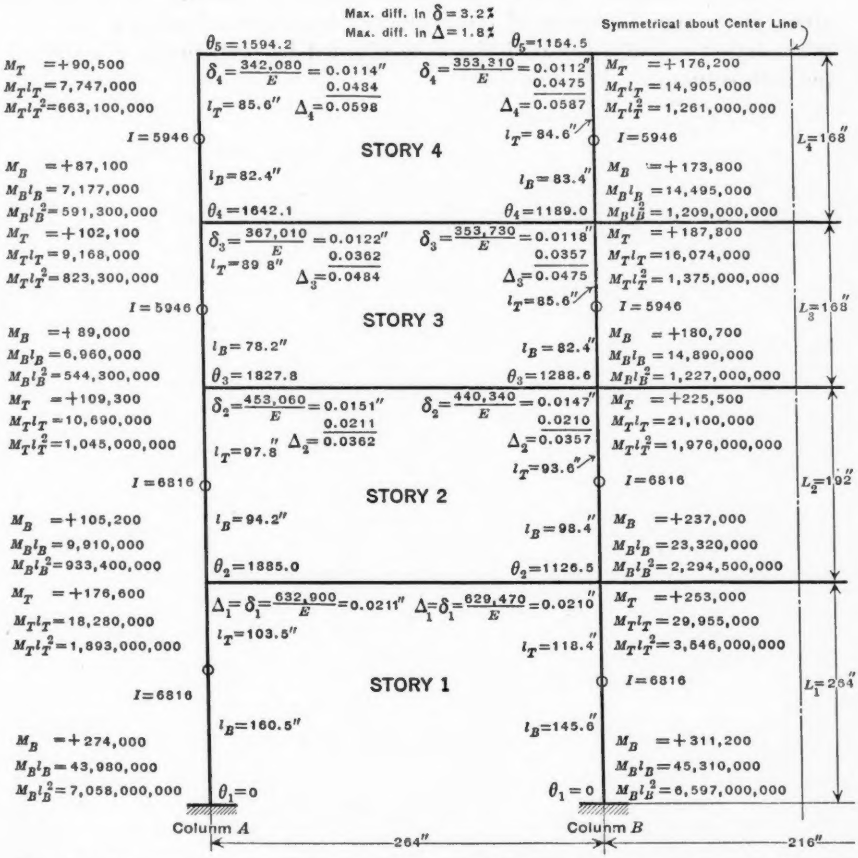

FIG. 6.—COMPUTATION OF DEFLECTIONS OF A WIND BENT WITH SHALLOW BRACING DUE TO COLUMN MOMENTS.

Equations (1) and (2) will enable the deflection of any story to be found quickly. The total deflection, Δ, from the base to any story will then be the sum of the individual deflections, δ, for the stories below.

The operation of computing the deflections from column moments in the manner thus outlined must necessarily start at the bottom of the building. If the twist angle, θ_B, for the first story is not zero, a solution may be effected, having regard to the probable degree of fixity at the base, by assuming some reasonable relation between θ_B and θ_T for each column in that story.

As an illustration of the application of the foregoing method, the deflections of the Wilson and Maney bent for the lower four stories, due to column moments, were computed at each floor and summed to show the total deflection from that floor to the base, as indicated in Fig. 6. Moments are those determined by the semi-graphical process indicated in Fig. 4 and are expressed in inch-pounds. Values of θ are indicated for $E = 1$, but deflections, δ, and Δ, are based on $E = 30\,000\,000$ lb. per sq. in.

While a separate deflection calculation for each column is not necessary, since, if the length change of the girders is neglected, all columns of a story must deflect equally, a useful check is afforded by performing the operation for each column.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

RUN-OFF—RATIONAL RUN-OFF FORMULAS

Discussion

BY C. E. GRUNSKY, PAST-PRESIDENT, AM. SOC. C. E.

C. E. GRUNSKY,[41] PAST-PRESIDENT, AM. SOC. C. E. (by letter).[41a]—As long ago as 1908 the writer attempted a mathematical demonstration of the propriety of using a rational formula for the determination of the maximum rate of run-off (maximum stream flow), based on the maximum rate of rainfall or maximum rain intensity. In the paper[42] describing that work it will be seen that as a basis for the mathematical presentation the fact was taken into account that the water which falls as rain—in any period of time that may come under consideration when the maximum rate of run-off at some selected point is to be estimated—must (at the peak of discharge) either have already passed the point in question or must still be in temporary storage: (1) As a film of water on the surface; (2) as water in conduits; (3) in watercourses; or (4) in lakes and reservoirs within the water-shed tributary to the point. Loss by evaporation in the consideration of small areas and short critical periods may be neglected or, rather merged with the other factors that go to make the maximum run-off rate less than the maximum rate of fall of water upon the water-shed as rain.

In the resulting analysis local records of heavy rainfall were necessarily taken into account and, of course, also the formulas for maximum rain intensity based thereon. These formulas determine the total quantity of water which may be precipitated upon any water-shed during the continuance of the rain which produces maximum stream flow.

Account was then taken of water-shed characteristics, such as extent, shape, topographic and orographic features, the degree of permeability of the surface, etc. These characteristics serve as a basis for approximating the critical time applicable to the water-shed; that is, the time within which a maximum

NOTE.—The paper by R. L. Gregory and C. E. Arnold, Associate Members, Am. Soc. C. E., was published in April, 1931, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: September, 1931, by Messrs. Le Roy K. Sherman, Francis Bates, and John W. Raymond, Jr.; November, 1931, by Messrs. Reginald A. Ryves, G. S. Tapley, W. I. Hicks, John M. Kemmerer, Carl H. Reeves, Leonard L. Longacre, G. H. Hickox and Donald M. Baker; December, 1931, by Albert R. Arledge, Assoc. M. Am. Soc. C. E.; and January, 1932, by Clarence S. Jarvis, M. Am. Soc. C. E.

[41] Cons. Engr. (C. E. Grunsky Co.), San Francisco, Calif.

[41a] Received by the Secretary December 5, 1931.

[42] "The Sewer System of San Francisco, and a Solution of the Storm-Water Flow Problem," *Transactions*, Am. Soc. C. E., Vol. LXV (1909), p. 294 *et seq.*

downpour would produce maximum stream flow. It was assumed that this time would be that period which would elapse from the beginning of the rain until all essential major portions of the water-sheds were contributing run-off to the stream at the point for which maximum stream flow is to be determined.

In 1930, the writer published simplified rain-intensity formulas.[43] These result in part from the substitution of the hour for the minute as the unit of time. These new formulas are intended to permit closer approach to actual fact, both for precipitation during very short time periods, $\frac{1}{2}$ hour, or less, and during very long periods. The suggested refinement for very short time periods is of value mainly when a short time record must be used in determining formula constants for any locality; otherwise, it is of no particular importance.

The intensity of rainfall is never uniform throughout any time period. It is not uniform at all points in a water-shed at any particular instant. Neither is the aggregate amount of rain during any period uniformly distributed. Nevertheless, the intensity of rainfall which will naturally be introduced into any rational formula for maximum storm-water discharge is the maximum average intensity throughout the water-shed, during the time which intervenes from the beginning of a storm of the type which will produce maximum discharge to the instant when this maximum discharge occurs.

This time, called for convenience "the critical time," must be determined, as already stated, from water-shed characteristics. When the water-shed has a compact, fairly regular form without long finger-like distortions, it will approximate the time that it takes water, in quantity, to flow from the remote parts of the water-shed to the point of concentration. Ordinarily, it must be left to the good judgment of the engineer to make a fair approximation of this period. This time as well as the rain intensity being in the nature of the case approximations, it becomes evident that great refinement in computation is less important than good judgment in evaluating basic elements. Frequently, in the case of large water-sheds the maximum discharge results from rain on a fractional part only of the entire water-shed.

It may be noted in passing that it is inadvisable to use any of the well-known older types of formulas for maximum stream flow, which do not include rain intensity as a factor.

The writer favors his own rain intensity and run-off formulas of which the former have already been presented to the Society.[44] For convenience they are here repeated with adherence so far as practicable to the author's notation:

For $H < 0.33$ hour (20 min.):

$$X_{\text{max.}} = 0.75\, K\, H^{\frac{1}{4}} \qquad (94)$$

$$i_{\text{max.}} = \frac{0.75\ K}{H^{\frac{3}{4}}} \qquad (95)$$

[43] *Monthly Weather Review*, October, 1930; and, *Military Engineer*, May-June, 1931; also, *Proceedings*, Am. Soc. C. E., May, 1931, p. 773.

[44] *Proceedings*, Am. Soc. C. E., May, 1931, p. 773.

For $H > 0.33$ hour and < 64 hours:

$$X_{max.} = K\,H^{\frac{1}{2}} \qquad (96)$$

$$i_{max.} = \frac{K}{H^{\frac{1}{2}}} \qquad (97)$$

For $H > 64$ hours:

$$X_{max.} = 2\,K\,H^{\frac{1}{3}} \qquad (98)$$

$$i_{max.} = \frac{2\,K}{H^{\frac{2}{3}}} \qquad (99)$$

If now the maximum rain in 1 hour be represented by R, instead of by the author's notation, $X_1\,(_{max.})$ it will appear from Equations (96) and (97) — because $K = X_1\,(_{max.}) = i_1\,(_{max.})$—that the value of K is the maximum rain in 1 hour and $K = R$. Equations (94) to (99), inclusive, may therefore be written:

For $H < 0.33$ hour:

$$X_{max.} = 0.75\;R\;H^{\frac{1}{4}} \qquad (100)$$

$$i_{max.} = \frac{0.75\;R}{H^{\frac{3}{4}}} \qquad (101)$$

For $H > 0.33$ hour and < 64 hours:

$$X_{max.} = R\,H^{\frac{1}{2}} \qquad (102)$$

$$i_{max.} = \frac{R}{H^{\frac{1}{2}}} \qquad (103)$$

For $H > 64$ hours:

$$X_{max.} = 2\,R\,H^{\frac{1}{3}} \qquad (104)$$

$$i_{max.} = \frac{2\;R}{H^{\frac{2}{3}}} \qquad (105)$$

When the value of $K = X_1\,(_{max.}) = R$ is to be determined for any area, the actual maximum precipitation on that area, during some period approximating the critical time, should be used as the basis for this determination. The necessary computation can then readily be made with the aid of the appropriate one of the three Equations (100), (102), or (104).

In any area of considerable extent the maximum average rain intensity on the entire area will be less than at single points of heavy rainfall. The maximum intensity is not coincident in time at all points of the water-shed. The precipitation of water upon a water-shed in the form of rain is at a more rapid rate than the run-off. Not all the water which falls upon a water-shed will reach the stream or other drainage conduit. Some of it sinks into the surface formation and ultimately either evaporates or remains underground so long before re-appearing in springs that its effect upon the maximum stream flow is lost.

Therefore, some modification factor with a value less than unity will have to appear in any formula for maximum stream flow if based on maximum rain intensity. Let α (the a in the writer's formulas published elsewhere) represent this modification or water-shed factor dependent upon surface characteristics and the extent of the water-shed; and let q represent the maximum run-off rate or maximum stream flow, in second-feet per square mile of water-shed area.

One acre-inch of rain is equivalent to $\left(\frac{43\ 560}{12} =\right)$ 3 630 cu. ft. of water. Consequently, if 1 in. of rain falls on 1 acre in 1 hour (= 3 600 sec.) and if all the water should run off at the rate of the rainfall, then this rate of run-off would be $\frac{3\ 630}{3\ 600} = 1.0$ cu. ft. per sec. In round numbers, therefore, 1 in. of rain in 1 hour on 1 sq. mile, or 640 acres, if all ran off in 1 hour, would be equivalent to a flow of 640 sec-ft.

Not all the rain which falls on a water-shed in the critical period, even when falling on an impervious surface, will appear as run-off during this period, or in any other period of equal duration. There will be some loss, as stated previously, and there will be a greater quantity of water in transit, that is, in temporary storage, at the end of any critical period than at the beginning of the period. It will suffice for practical purposes to take this reduction in the quantity of discharge compared with the quantity of rain water at about 25% and to assume, therefore, that actual run-off per square mile in the critical period, when the rain intensity is 1 in. per hour, will be about (75% of 640 =) 480, or, in round numbers, say, 500 sec-ft. per sq. mile instead of 640.

Because the value of X_1 (max.), or R, which represents the maximum rainfall on the water-shed in 1 hour, may be substituted for the coefficient, K, and because t, expressed in hours, is now to be written for H, the time of concentration (the critical time), the formulas for maximum run-off per square mile will take the form:

For $t < 0.33$ hour:

$$q = \frac{375\ \alpha\ R}{t^{\frac{3}{4}}} \qquad \text{(106)}$$

For $t > 0.33$ hour and < 64 hours:

$$q = \frac{500\ \alpha\ R}{t^{\frac{1}{2}}} \qquad \text{(107)}$$

For $t > 64$ hours:

$$q = \frac{1\ 000\ \alpha\ R}{t^{\frac{2}{3}}} \qquad \text{(108)}$$

If A_m represents the area of the water-shed, in square miles, and $Q_{max.}$, the maximum rate of discharge, in second-feet, then:

For $t < 0.33$ hour:

$$Q_{\text{max.}} = \frac{375 \ \alpha \ R \ A_m}{t^{\frac{3}{4}}} \quad \text{.....................(109)}$$

For $t > 0.33$ hour and < 64 hours:

$$Q_{\text{max.}} = \frac{1\,500 \ \alpha \ R \ A_m}{t^{\frac{1}{3}}} \quad \text{.....................(110)}$$

For $t > 64$ hours:

$$Q_{\text{max.}} = \frac{1\,000 \ \alpha \ R \ A_m}{t^{\frac{2}{3}}} \quad \text{.....................(111)}$$

In some cases, the value of the coefficient, α, may be determined from known (measured or closely approximated) high rates of discharge and the rainfall characteristics which produced these rates. Generally, however, assumptions must be made relating to this value. It appears obvious that of the total quantity of water falling on a water-shed as rain, the relative quantity which evaporates, together with that which sinks into the ground and is thus prevented from contributing to the peak discharge, increases as the critical time (that is, as area) increases.

This consideration has led to the tentative adoption of the following expression for α and the selection of values for surface characteristics which may be used until something better is offered for typical localities:

$$\alpha = \frac{1}{1 + \beta \sqrt{t}} \quad \text{........................(112)}$$

in which, β is a factor dependent on the surface conditions of the water-shed. For the present:

$\beta = 0.013$ for impervious areas.
$\beta = 0.25$ for mountains.
$\beta = 0.40$ for rolling country.
$\beta = 1.3$ for flat country (ordinary soil).
$\beta = 6.5$ for sandy regions.

These values are suggested for ordinary conditions in temperate climates. The resulting values of the coefficient, α, will be found, no doubt, too small in localities where the ground may be frozen, where it is water-logged, or where the maximum run-off occurs when heavy rains fall on snow.

The intensity of rainfall under extreme conditions is now fairly well known throughout the United States for individual stations. It is not so well known what the maximum rainfall in a limited time period may be throughout water-sheds of considerable extent. The rain intensity at single stations may quite properly be accepted as applying to small areas. For large areas the mean maximum rain intensity will be materially less than the maximum rain intensity on small areas.

For convenience, it may be assumed that for a critical time period of 1 hour, or less, the rainfall throughout the entire water-shed under consideration will be of the same intensity as at single stations.

For longer critical periods and larger water-sheds the maximum rain intensity (computed as if the rain were distributed uniformly over the area in question) will be based on the best meteorological data available. Its deter-

mination throughout any water-shed in question during the critical time, t, is best made from actual records of heavy rainfall and a study of its distribution to the various parts of the water-shed. Such records, however, will rarely, if ever, indicate the true possible maximum. Consequently, some correction factor greater than unity must be applied, based on a comparison of the individual station records during such a storm with the possible local maxima. It should not be assumed, however, that rain of maximum intensity will occur at all the stations at the same time, but rather that if the storm in question at controlling stations might have produced an intensity greater by some percentage, then it is proper to assume that with similar relative distribution of rain throughout the water-shed some future storm might produce the same proportionally greater amount of rain and of run-off.

Due to topographic and orographic characteristics of the water-shed, coupled with the distribution of the rain throughout its extent, it will frequently be found, as already stated, that a rainfall of maximum intensity on a part of the water-shed during a relatively short critical period will produce a greater peak discharge than that which occurs when the rain of maximum intensity for the longer critical time applicable to the entire water-shed is taken into account. Trial computations alone, in the light of rainfall data, can determine what part of the river system is to be excluded in making the estimate of maximum momentary river discharge.

The method of estimating the maximum stream flow can now be summarized briefly as follows:

(*a*) Determine the critical period for the water-shed, with a proper allowance for the time that it will take water to flow in the stream from the water-shed to the point at which maximum stream flow is to be ascertained. Express this critical time, t, in hours.

(*b*) Determine the value of the water-shed coefficient, α, from the surface characteristics of the water-shed by use of Equation (112).

(*c*) Determine from the rainfall records, the maximum depth of rain, expressed in inches, which has fallen upon the water-shed in any period, about t hours in duration, and with the aid of single station records by means of a correction factor establish the probable maximum rainfall on the water-shed in t hours. This maximum and the critical time will establish the maximum intensity for the water-shed and this, in turn, will be used in calculating the value of R ($= X_1$ $(_{\text{max.}})$) applicable thereto. The contribution by the authors on this subject of a determination of the critical time should be found helpful.

(*d*) Use Equations (106), (107), or (108), as the case calls for, to determine the maximum flow, in second-feet per square mile, or Equations (109), (110), or (111) to determine the maximum stream flow, in second-feet, at the point under consideration.

It remains to be said that lakes and reservoirs are frequently factors which prolong the critical period and reduce the maximum stream flow. Ordinarily, their effect is fairly well taken account of in the estimate of the duration of the critical period. When, however, the point for which maximum flow is to

be determined lies close below a large lake or reservoir, recourse may have to be had to a special study of the effect of the increase of storage in such a water body upon the discharge at the peak of the flood flow.

The following examples will illustrate the application of the formulas:

Example (*1*).—Let it be supposed that for a mountainous water-shed of 2 000 sq. miles, the critical time, that is, the value of t, has been approximated at 36 hours. The most severe storm of which there is record, as indicated by a number of rain gauges and by the resulting isohyets, produced an average of 4 in. of rain throughout the water-shed during this period. However, it is known that all single-station records for this storm have been, or may have been at some time, exceeded by 50 per cent. What should be assumed as the probable maximum stream flow?

Because single-station records have been exceeded it is probable that the maximum measured rain intensity throughout the water-shed should be increased in like proportion. Instead of 4 in., therefore, 6 in. should be introduced in the calculation. Equation (107) will apply.

It follows that: $i_t = \frac{6}{36} = 0.167$ in. per hour, the average for the 36 hours;

and $i_t = \frac{R}{\sqrt{36}} = 0.167$. Therefore, $R = 1$ (determined in this case directly from the rain on the entire water-shed; not from any single-station record).

From Equation (112):

$$\alpha = \frac{1}{1 + 0.25\sqrt{36}} = 0.40$$

and from Equation (107):

$$q = \frac{500 \times 0.40 \times 1}{\sqrt{36}} = 33 \text{ sec-ft. per sq. mile}$$

and,

$$Q_{\text{max.}} = 2\,400 \times 33 = 80\,000 \text{ sec-ft.}$$

Example (*2*).—In a down-town area having an extent of 20 acres, or 0.0313 sq. mile, 70% of the area is impervious; the remainder is ordinary soil. The critical time is estimated at 15 min. (= 0.25 hour). The maximum rainfall in 1 hour is known to be 3 in. What will be the maximum rate of run-off from this area? Use Equation (106). Here,

$$R = 3;\ i_{\text{max.}} = \frac{0.75\,R}{0.25^{3/4}} = \frac{2.25}{0.354} = 6.3 \text{ in. per hour}$$

and,

$$\alpha = 0.79\,\frac{1}{1 + 0.013\sqrt{0.25}} + 0.30\,\frac{1}{1 + 1.3\sqrt{0.25}} = 0.88$$

$$q = \frac{375 \times 0.88 \times 3}{0.25^{3/4}} = 2\,830 \text{ sec-ft. per sq. mile}$$

and,

$$Q_{\text{max.}} = 0.0313 \times 2\,830 = 89 \text{ sec-ft.}$$

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DISCUSSIONS

FINANCING STREET AND HIGHWAY IMPROVEMENTS

Discussion

By J. C. Carpenter, M. Am. Soc. C. E.

J. C. Carpenter,[8] M. Am. Soc. C. E. (by letter).[8a]—The problem involved in this paper is the determination of the most equitable method of obtaining highway funds in such a manner that the burden may be adjusted according to ability to pay and the funds distributed according to the needs of traffic. This determination has been made customarily by legislative bodies with very little advice from engineers. Several years ago a legislator made a public statement that it is the function of the Legislature to decide on the amount of funds necessary as well as to determine the method of collection, and all that will be expected of the engineer is proper expenditure.

It is becoming more and more evident that the engineering and financing are so intimately connected that they cannot be considered separately, and engineers are being called upon for advice as to methods of financing. When times are good the average citizen is too busy to inform himself on routine governmental action. He grumbles about payment of an excessive tax, but does not bother to look into the use that is made of his funds. When times are not so good, however, the taxpayer takes an active interest in knowing just what use is made of his hard-earned dollars and also inquires as to the functions performed by those entrusted with his money. Equitable collection of funds and proper distribution of the finances so as to furnish satisfactory service for those who provide these funds are necessary if the public is to remain satisfied with results.

The classification of highways into the three groups outlined by Mr. Crum is commonly accepted. The primary roads, as defined, cover the State highway systems as now laid out. The Federal Aid Act provides for a system not

Note.—The paper by R. W. Crum, M. Am. Soc. C. E., was published in August, 1931, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: In October, 1931, by Messrs. A. P. Greensfelder and C. R. Thomas.

[8] Fort Worth, Tex.

[8a] Received by the Secretary December 28, 1931.

to exceed 7% of the public highway mileage of the State, at the time of the passage of the Act, and divides the highways thus designated into primary (not to exceed three-sevenths of the system) and secondary. Since this law was passed the American Association of State Highway Officials has mapped out a system of numbered highways extending across the country in all directions. The established primary highways in all cases, do not follow the numbered highways, and there are a few isolated instances in which the numbered highway may not be a part of the Federal Aid System. The Association, in establishing this system of marked highways, performed a service of inestimable value to the traveling public. In the writer's opinion, the highways on this system should be considered the primary highways of the country.

Secondary highways should include those of enough importance, outside of the primary system, to be included in the State highway system. Any highway within the description given by Mr. Crum for secondary highways should be a part of the State highway system.

Tertiary highways, or third-class roads, should include all other necessary public highways, except city and village streets. If a highway is open to public travel and serves one or more individuals, it should be included in the classification.

Changing conditions make reclassification necessary as is obvious from the many changes in the Federal Aid System since the first system was adopted after the passage of the Federal Road Act in 1916. A highway that is in the secondary class to-day may be a primary highway five years from now, or *vice versa,* by change of traffic trend, a primary highway may become a secondary or even a tertiary highway to-morrow. Nevertheless, with careful planning and far-sighted determination of the location of routes, the systems will finally become established and the changes which could not be foreseen in the early days of highway planning will become much less extensive and of minor importance. This condition is eminently desirable from the standpoint of economical expenditure of highway funds.

The problem of determining just what portion of the cost should be paid by each individual on a complete highway system for the entire country is by no means simple. Consider the case of the farmer who lives at the end of the third-class road. When he leaves his garage he travels to the farm gate over a right of way which is reserved for his special use to the exclusion of all others. Obviously, he should pay the entire cost of any improvement on this location; but when he passes his gate and enters the third-class highway he may mix with trucks hauling farm products to the community center, tax assessors from the county seat, State officials on business for the Commonwealth, and the United States mail carrier on his regular round of duty. He may legitimately expect this traffic to help him finance the improvement and maintenance of the roadway in proportion to the service it receives. After he emerges into the secondary system he finds a well-paved road which may be provided by the State, and this service may be of more value to him than the service his tertiary road affords to the occasional State officials who may use it. If he visits his cousin in the next State he uses the primary highway system. With Federal Aid on this highway this farmer is receiving some

return for the use of his tertiary road by the United States in carrying the mail. This simple example illustrates the complications that surround a solution of the problem of financing and service.

During 1930 almost $2 000 000 000 were provided for State highways and local roads. A study of available statistics for 1931 indicates that there is much variation in the amounts collected and the methods of distribution in the different States. There are almost as many different schedules of license fees as there are States. Gasoline tax rates run from 2 cents to 6 cents, with an average of 3.35 cents for the entire country. One of the States with the lowest rate of 2 cents has the largest total mileage of a high type of surfaced highway. The income for State highways was $1 136 673 437 and for local roads, $818 379 508.

The following figures are for the entire United States. There are many variations for the individual States, but these divisions are representative of the entire situation as far as collections and distribution are concerned. For State highways the total funds were derived in the following manner:

	Percentage
State tax levied	1
Appropriation by States	2.9
Motor vehicle fees, etc	25.5
Gasoline tax receipts, etc	36.2
Miscellaneous income	1.5
State bonds and notes	19.5
Federal Aid post roads fund, allotment used	8.1
Funds transferred from local authorities	5.3

State highway disbursements total $1 139 676 601, and disbursements by local authorities, $851 686 625. State highway disbursements were distributed as follows:

	Percentage
For construction and right of way	62.5
For maintenance	16.9
For miscellaneous expenses	0.2
For equipment and machinery	2.0
For interest payments on bonds and notes outstanding	4.4
For principal payment on bonds and notes	6.1
For transfer to county or town for local roads	5.9
For other obligations assumed	2.0

These figures show that construction and right of way absorbed the equivalent of all receipts from Federal Aid, State bonds and notes, motor-vehicle fees, and 9.4% of other receipts. After deducting this 9.4% from the gasoline tax receipts there remains 26.8%, which is almost enough to pay the total cost of maintenance and the principal and interest payments on bonds and notes.

With the increase in constructed mileage of the State systems there will be, of course, a reduction in the percentage spent for construction and an increase in the percentage paid for maintenance.

The grand total of collections from gasoline taxes for 1930 was $494 683 410. This was distributed, as follows:

	Percentage
Collection cost	0.2
Construction and maintenance of State highways.	68.5
Construction and maintenance of local roads	19.5
State and county road bond payment	6.3
For miscellaneous purposes	5.5

The total for miscellaneous purposes is $27 378 986. It is encouraging to note that $13 509 516 is transferred to cities for expenditure on streets, aviation, free bridges, highway administration, and refund reserve. These amounts are expended for purposes indirectly related to rural highway traffic. Some States recognize, in this way, that city streets carrying highway traffic are entitled to a portion of the revenues from gasoline taxes. For school purposes Texas, Florida, and Georgia divert $13 423 880, or 2.7%, of the entire fund collected; and an additional 0.1% is used for purposes that have no connection with highways. While it is generally admitted that the diversion for schools of $7 381 774 from the Texas gasoline tax receipts is not equitable, there is some consolation in the fact that the State Highway Department receives in transfers from local authorities $10 037 600. The $355 704 860 received for license fees, registrations, etc., is distributed as follows:

	Percentage
For collection and administration	5.4
For State highways	62.4
For local roads	19.3
For State and county bonds	10.2
For other purposes	2.7

The larger part of the 2.7% for other purposes is expended for motor patrols. This tabulation indicates practically the same major distribution as for gasoline taxes.

There is need for a thorough study of all legislation providing funds for highway work, such as is now being undertaken by a committee of the American Road Builders Association. This will be valuable in promoting uniform laws throughout the country. The results of such a study when presented to a legislative body would be convincing and no doubt would be welcome to those who formulate the laws. The Tax Commission of Wisconsin has been authorized under an Act of the Legislature of 1911, to collect information on tax assessment, receipts, and expenditures, and the reports have been found exceedingly valuable by legislatures and economists.

The 1930 joint survey of the North Carolina State authorities and the United States Bureau of Public Roads is a valuable contribution toward the solution of problems of financing. This indicates that all *ad valorem* taxation is not a just method of collecting funds for highway purposes. It also makes clear that the county unit is unsatisfactory as an administrative division. Among the 3 000 or more counties of the United States, there are numerous instances of competent engineering and administrative organization, but almost one-quarter of all the counties have no definite, organized, engineering

control over their highway work. The centralization of highway authority for several counties would undoubtedly result in more efficient and economic expenditure of funds.

Before the annual cost of all or parts of any State highway system can be calculated there must be careful study of the adequacy of the system as at present adopted. The systems now in use (1932) were laid out about 1922 and were generally well suited for the needs of the various States; but in ten years there has been a marked change in traffic conditions, industrial and agricultural communities, etc. In addition, it was necessary, in the early days of highway construction, to locate the highways so as to meet the needs of individual communities of small size. With the "nation on wheels" the public is now interested in, and demands, direct routings, good alignment and grades, smooth surfaces, and freedom from congestion. The highway engineer can now, with public support, use correct engineering principles in location.

A revised system should be laid out in such a manner as better to serve local and through traffic for the entire country. It is important that such a system be developed at an early date. Based on present and probable traffic trends, on the development of the industrial and agricultural sections of the country, the growth of cities, and the controlling topographic features, a careful study should be made in each State and a major system laid out and filed for use on new work. It will be necessary to consider, carefully, entirely new locations, extending across the State, in some instances. The location through and adjacent to cities, the distance between the highways and other utilities, such as railways, etc., should be carefully determined in considerable detail.

While it would be impossible to change at once from the present system to an entirely new one, the data should be collected and the system developed and placed on file with the expectation of utilizing the ideal locations for construction of parts of the system as funds become available. Under the present policy, large outlays of money are required to reconstruct old projects on narrow rights of way with undesirable alignment, and traffic congestion is increased, rather than diminished. With these same funds a new highway can often be built on good alignment, with adequate width of right of way for future development, and with much better planned surfacing, leaving the old road to serve local traffic, with a much smaller maintenance cost for the two routes.

The new location serves and develops an additional local community, and also provides ample room, at low ground rental, for commercial activities necessary for highway traffic (such as garages and hotels) without interfering with established stores and other services essential to the community, on the old road.

Most of the State highway departments now have a large volume of information on costs of all the elements of highway construction and maintenance. After having laid out the system, it should be possible for a department to work out the cost of construction and with it the annual cost of systems to the States.

There are in excess of 2 400 000 miles of local roads in the United States. Not more than 300 000 miles have been graded, drained, and surfaced. With the increased demand for "farm to market roads" and consequent increase in the amount of funds being expended for local roads, many more of them will be improved and surfaced. Logical co-ordinated planning is essential for economical and efficient expenditure of these funds. In addition to combining and re-routing local highways so as to furnish the best service for the fewest miles, the well planned system will be comprehensive and will be based on proper consideration of such factors as correct utilization of farm lands (such as prospective abandonment of some farm lands which are being worked without profit and probable change in population density after a soil survey to establish correct utilization of the land), re-adjustment of so-called marginal or sub-marginal lands, etc.

The information and plans established by regional planning boards and commissions should be correlated with the plans for the system of primary, secondary, and tertiary highways. The establishment of regional planning areas in rapidly growing communities is desirable. Any form of governmental organization which tends to ascertain regional requirements officially is better than no organization at all. Such an organization will create an informed body of public opinion on the subject. The success of regional planning depends on the extent to which those who furnish the funds are convinced of its practical utility. Enabling acts are desirable in those States where regional planning has not been sanctioned by law.

Traffic studies are essential in the planning of any system of highways. Many States have traffic records extending over a period of years and find them valuable in prophesying future construction. A study of city streets traversed by rural traffic, compared to that on other streets of the city, should furnish a basis of determining the portion of the cost of improvement to be paid by rural traffic.

A study of real estate values as affected by road improvement would prove valuable, but there are so many other elements which change values that it is difficult to apply any factor of assessment derived by this means. The existing governmental agencies organized for the purpose of assessing property should be utilized to furnish these data and any adjustments in values should be determined by the regularly constituted assessors. There is no doubt that improved highways change property values and those benefited should pay a part of the original cost, but as previously stated, the North Carolina survey indicates that the *ad valorem* tax is not an equitable method of financing all improvement costs.

The necessity for careful and extensive planning of locations for highways is much more obvious than ever before. In issuing bonds for highways it has been considered sound to assume that the location, grading, and drainage structures are permanent features, but numerous instances are found in which modern traffic demands locations which mean the abandonment of considerable sections of these old roads. The planning cannot stop at county lines, but must be extended to develop locations which will not be subject to change.

The magnitude of the problem is difficult to grasp, because there are many elements to be considered in proper planning, and failure to include all essentials will mean that the entire scheme will be temporary rather than permanent. This is a certainty: Planning must precede any scheme of financing and only such financing programs as are based on a carefully worked out plan will be successful and result in satisfaction for the individuals paying for the work.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
DISCUSSIONS

DESIGN OF LARGE PIPE LINES

Discussion

BY MESSRS. C. M. ORR, AND EDWARD J. BEDNARSKI

C. M. ORR,[33] ESQ. (by letter).[33a]—Reference has frequently been made to the bending stresses in the rim zone of a pipe line due to restraint of the pipe at the support. It has been customary to say that these stresses "shall be considered in the design," or "cannot be neglected in the design," although cases of similar restraint may be pointed out in which the effects of the restraint are neglected. While the existence of these stresses, approaching 1.82 times the $\frac{pr}{t}$ cylinder stresses, is not questioned, it may be argued that they may be neglected as logically or safely in the case of a pipe shell as in the case of the following:

(1) In the shells of vertical, flat-bottom storage tanks, near the connection to the bottom, the bending stresses approach the theoretical 1.82 $\frac{pr}{t}$ value; yet it is in accordance with standard specifications to consider only the $\frac{pr}{t}$ tension stresses in the design.

(2) In the shells of elevated, suspended bottom tanks, near the connection to the bottom, bending stresses are introduced, due to the restraint of a horizontal circular girder. This girder is necessary to resist the inward thrust from inclined tower posts, but its effect on shell stresses is commonly neglected, it being usual to consider only $\frac{pr}{t}$ tension stresses in the shell design.

(3) In the case of every pressure container composed of a cylindrical shell with closed ends, the shell is restrained to some extent by the ends, yet the design is based on stresses due to pressure.

NOTE.—The paper by Herman Schorer, Assoc. M. Am. Soc. C. E., was published in September, 1931, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: November, 1931, by Messrs. L. J. Mensch and W. P. Roop; December, 1931, by Messrs. Johannes Skytte, Donald E. Larson, Raymond J. Roark, and F. W. Hanna; and January, 1932, by Messrs. Paul Bauman and L. J. Larson.

[33] Asst. Research Engr., Chicago Bridge & Iron Works, Chicago, Ill.

[33a] Received by the Secretary January 4, 1932.

While it is recognized that, in some respects, the foregoing examples are not identical with the pipe line, they are presented with the idea of suggesting that bending stresses due to restraint are not necessarily of prime importance in pipe-line design.

The combination of rim bending stresses with longitudinal stresses due to beam moments would affect only a small section of the pipe. The effect of overstressing the pipe by this combination would be some local change of shape affecting a small area, rather than failure of the pipe as a whole. If this change of shape were sufficient to cause a permanent distortion over this small area, it would be of such small extent and relatively small magnitude as to be negligible. Common prectice permits not only the use of material previously stressed beyond its yield point, but it also permits working stresses in this material comparable with stresses in material which has not been previously distorted.

Complete elimination of all bending stresses in the rim zone due to restraint may be imagined. If it were practical to make the diameter of the ring equal to that of the pipe when expanded by full pressure, and then to stretch the shell plate at the ring to this increased diameter, leaving the remainder of the shell unchanged, no rim bending would occur under full load.

If it were possible to support the shell at the ring by radial members projecting from the shell through holes in the ring, in such a manner that expansion of the pipe under pressure were permitted, no rim bending stresses would occur.

Either of these suggestions would probably be impractical, but the writer mentions them with the idea of emphasizing the thought that they are also unnecessary. Because similar restraint stresses are safely ignored in other structures, and because of the limited area of pipe affected by such stresses, and the small possibility of serious distortion over this area, may not these stresses be neglected in large pipe lines?

EDWARD J. BEDNARSKI,[34] ASSOC. M. AM. SOC. C. E. (by letter).[34a]—The author has presented an interesting study of the reaction exerted by a thin-walled pipe ($\frac{r}{t}$ = 200 and more) on a support in the form of a rigid ring or a diaphragm when the pipe is horizontal and precisely full. In Conclusion 2 he states that,

> "If the pipe is supported by stiff, disk-shaped members * * *, the shell is entirely stable without any intermediate stiffening rings, thus permitting the use of large spans in combination with thin plates."

The "Complete Report on Construction of the Los Angeles Aqueduct" contains the statement that $\frac{1}{4}$ in. is the minimum thickness of steel plate that can be used for pipes as large as 10 ft. in diameter, and still maintain a satisfactory rigidity. This limit, it is claimed, gives satisfactory tensile strengths to sustain a head of as much as 144 ft.

[34] Civ. Engr., Sacramento, Calif.

[34a] Received by the Secretary January 7, 1932.

Before it was filled, the Nine-Mile Siphon (designed diameter, 9.5 ft.), of the Los Angeles Aqueduct[35] measured 9 ft. vertically and 9.81 ft. horizontally. Under a pressure of 50 lb. per sq. in., these inside dimensions changed to 9.54 ft. and 9.58 ft., respectively.

The pipe was placed on concrete piers, 2 ft. wide, spaced 24 ft., center to center, and cast to fit the pipe. When two adjacent piers on a side hill, settled away from the pipe, the span was 72 ft. This pipe when filled remained suspended without injury until the defective supports could be replaced.

In another instance[36] a break occurred in a section of the aqueduct composed of a 10-ft. pipe, with $\frac{1}{4}$-in. and $\frac{5}{16}$-in. shells, crossing the Antelope Valley in the Mojave Desert region. The water escaped rapidly, under a head of 200 ft., and the pipe collapsed like an emptied fire hose for a distance of nearly 2 miles. At some points the top of the pipe was forced down to within a few inches of the bottom. After the break was mended the water was turned on and the pipe restored to its original form by pressure.

In the example given by the author the pipe has all the characteristics of the Los Angeles Aqueduct line, with a pressure of almost 50 lb. per sq. in. Therefore, it might possibly stand up for some time on similar piers, with a span of 60 ft.

The property of this kind of a pipe, when empty, to change its shape to another cross-section when under pressure, proved to be an asset. The thin-walled pipe cannot stand the bending stresses any more than the cables of a suspension bridge. It seems that under a liquid load of comparatively light specific gravity a thin-walled circular pipe with a comparatively high modulus of elasticity, just full, cannot assume any shape except that of a circle (regardless of the method by which the dead weight of the pipe and the liquid is supported), because a circle is a contour containing a maximum possible area for a given perimeter or a contour of a maximum expansion. As any other shape contains less area, a pipe in order to take this other shape would have to expel a part of the liquid it contains. If it is assumed that the water cannot escape from the pipe, the pipe may change the form of its cross-section to another form containing the same area only by elongation of its perimeter. For a very small elongation a distortion consequently should be insignificant. In the case of a thin-walled rubber pipe filled with mercury the shape of the pipe will change quite considerably with every particular method of support. It is probable, that for the ring-like support at the ends, the rubber pipe will retain a circular cross-section throughout its length.

Due to the flexibility of the pipe walls the bending stresses should be insignificant. Any additional internal pressure will add to the stability of the circular shape of the pipe and make it more nearly a perfect circle. In the case of a 10-ft. pipe, a head of 100 ft. makes a difference of only 10% between the pressures at the top and at the bottom, or 4.33 lb. per sq. in., as compared

[35] "Complete Report on Construction of the Los Angeles Aqueduct," Dept. of Public Service, Los Angeles, Calif., 1916, p. 195.

[36] *Loc. cit.*, p. 29.

with the total of 43.3 lb. per sq. in. This is not enough to produce any considerable distortion. In the foregoing case the distortion is $\frac{1}{2}$ in. for 50-lb. pressure on a pipe 9 ft. 6 in. in diameter. It should be noted that the pipe in this case was supported by concrete piers "2 ft. wide, and only 3 or 4 ft. of circumference of the pipe was bearing on the pier."[37]

The supports of the Los Angeles Aqueduct type seem to have the following advantages:

1.—They do not require special reinforcement for the pipe at the pier due to the difference between the rigidity of the pipe and the supporting ring, causing the additional bending stresses near the ring.[38] Thus, the pipe may be made of plates with uniform thicknesses for its entire horizontal length.

2.—The location of the piers may be changed if necessary in the field without causing any inconvenience.

3.—Uniform thickness of the pipe wall insures the pipe from being damaged beyond recovery in case a sudden vacuum occurs in the line and the relief valves fail to act promptly and efficiently.

The disadvantage of the Los Angeles Aqueduct type of pier seems to be the necessity of using shorter spans of 24 ft., as against 60 ft. for the ring-girder supports. It would be interesting to know what would happen to the Antelope Pipe Line in the Mojave Desert region if it were supported by ring girders and piers.

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[37] "Complete Report on Construction of the Los Angeles Aqueduct," Dept. of Public Service, Los Angeles, Calif., 1916, p. 196.

[38] See "Applied Elasticity," by Timoshenko and Lessels, p. 146, Westinghouse Technical Night School Press, East Pittsburgh, Pa., 1925.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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DISCUSSIONS

CONSTRUCTION MANAGEMENT

Discussion

BY MESSRS. EDWARD W. BUSH, AND A. P. GREENSFELDER

EDWARD W. BUSH,[3] M. AM. SOC. C. E. (by letter).[3a]—No doubt members of the Society feel greatly indebted to the author for this instructive paper. It deals with questions about which little has been written, but concerning which much can be said.

The author points out that "it is much better to let machinery depreciate, and parts of an organization stay idle, than it is to lose cash from a bank account"; also, that "most contractors do too much work rather than too little. The financial strain and lack of proper attention, both to general planning and small details, are generally harmful to profits under such conditions." The files of any large surety company will disclose cases without number which affirm the correctness of these statements. Bankers as well as surety underwriters recognize over-extension as one of the leading causes retarding the financial advancement of contractors. Analytical studies of the technical records and financial information of contractors, such as the surety underwriters are constantly making in their daily work, will disclose that advancement in financial strength is more likely to occur when the contractor makes a gradual increase in activities, rather than when he is trying to become a big contractor within a short period of time. The author has stressed the importance of a good superintendent and organization, and these are best developed by the contractor through a moderate expansion.

The contractor must be an optimist to be successful, as the business demands that he have the utmost faith in his own ability to perform, and in many cases it is this optimism which leads him to over-extend. Especially is this likely to occur after getting the "breaks" on a few contracts and thus "cleaning up" some nice profits. Under such conditions the business of contracting looks very attractive indeed and it is only a strong-minded

NOTE.—The paper by Richard Hopkins, Esq., was presented at the meeting of the Highway Division, New York, N. Y., January 16, 1930, and published in September, 1931, *Proceedings*. This discussion is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

[3] Engr., Aetna Casualty & Surety Co., Hartford, Conn.

[3a] Received by the Secretary November 30, 1931.

person who can resist the temptation to load up with about everything that can be taken. Many contractors have traded net quick assets or working capital for new construction plant to find to their sorrow that there was nothing left with which to operate. A contractor may be "plant poor" and not fully realize it until he tries to keep all of it at work.

A. P. GREENSFELDER,[4] M. AM. SOC. C. E. (by letter).[4a]—There is a fair difference of opinion, particularly in this time of depression, as to how much a construction organization should specialize or diversify its bidding. Specialization is seemingly the order of the day. An organization with specialized machinery and crew, theoretically, should have the advantage over one that does not. On the other hand, unless there is a great deal of that particular class of construction work continuously at hand in the community, it means that the organization must enlarge its territory and follow such class of work wherever it may be found. Diversification, on the contrary, permits a smaller radius of travel and guards against having all one's organization and capital tied up in one particular type of work.

Mr. Hopkins' comments are sound relative to percentage of work secured from the volume bid. Post bidding comments always indicate that the marginal loss between the low and the next low bid is the hardest money to regain actually. The statement is correct that one should not undertake too great a volume of work for the capital of the company. Over-eagerness in this respect has too often proved fatal.

It would seem vital that contractors, together with engineers, should evolve practical measuring rods for determining the relation between volume of work, quick and deferred assets, and construction equipment for various classes of work. The Bureau of Contract Information, Incorporated, set up jointly in Washington, D. C., by surety companies, is determining the performance record of contractors throughout the country. It should presently have sufficient experience rating to serve as a guide in making these determinations.

Unbalanced bidding is another feature which should be guarded against. One is likely to feel that his practical experience will warrant certain manœuvering of unit bid prices. There is usually enough gambling in construction without this and it is only warranted through the viewpoint of equity of the owner when the engineer's quantities are themselves unbalanced (which is, unfortunately, quite often the case). Engineers too frequently guess the quantities of excavation classification instead of predetermining them by making borings.

All estimates should include equipment rental or there will soon be no equipment to rent to oneself. Depreciation of construction equipment necessarily goes on at a very rapid rate. Obsolescence also plays its part. American inventors are daily producing new equipment so much more efficient than previous machinery that construction equipment five years old is frequently obsolete.

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[4] Pres., Associated Gen. Contrs. of America, Inc.; Pres., Fruin-Colnon Contr. Co., St. Louis, Mo.

[4a] Received by the Secretary, December 18, 1931.

In planning the work, contractors should realize the value of time studies. The current studies of the U. S. Bureau of Public Roads indicate that these are indeed worth while. Lost time can be minimized when one realizes the extent.

The increasing prevalence of including engineers in contractors' organizations is noteworthy. Such engineers, on the other hand, must realize the need of being practical as well as theoretical. Regardless of cost brains are the cheapest purchase an organization can make. The analytical mind of the engineer is essential to proper cost keeping, provided such cost-keeping furnishes, daily, the current trends of costs.

Mr. Hopkins very truly stresses the need of having on hand repair parts of machinery. Time lost awaiting repairs is expensive when a large organization is delayed. On the other hand, contractors are too apt to think in terms of labor-saving devices. They should analyze their jobs and consider machine-saving costs by not over-equipping them. Too frequently they use theoretical costs and realize to their sorrow that they have not found the operator whose hands fit the levers, or whose mind is sufficiently quick thinking or his body sufficiently active to make the operation of that machine profitable.

The preparation of advance schedules of work is essential. Such schedules serve as guides to the superintendent in the field as well as to the "boss" at his desk. They also furnish the necessary check as to the average performance. One hears of the wonderful day's over-run when records are broken, but not of the many days of under-run when machinery or organization is broken.

Construction management is rapidly becoming more and more scientific. It takes mind and matter jointly to produce smooth running construction jobs.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

SOIL MECHANICS RESEARCH

Discussion

BY MESSRS. C. H. EIFFERT, A. A. EREMIN, F. N. MENEFEE, AND E. G. WALKER

C. H. EIFFERT,[30] M. AM. SOC. C. E. (by letter).[30a]—Since the completion of the dam of the Miami Conservancy District the plan of exploring the cores of one or more or these dams had been under consideration. The desire to make such an exploration was prompted by the lack of knowledge of core conditions in the completed structure and by the difference of opinion among prominent engineers as to the exact nature of the core material some time after the completion of a dam. It was maintained by some that the core remained in a semi-liquid condition for a long period, while others were of the opinion that there might be a possibility of the core drying out and cracking to such an extent as to destroy its imperviousness. That the cores of the Miami Conservancy Dams did not long remain in a fluid or semi-liquid condition was indicated by the Goldbeck pressure cell readings and by some of the exposures made during construction.[31] These readings showed that the core material possessed considerable inherent stability at that time. Further knowledge as to the nature of the cores was not available until the recent experiments were made.

One reason for not making explorations prior to those under consideration was that facilities for testing samples and experimenting with the materials to the best advantage were not available. When it became possible to enter into a co-operative agreement with the Massachusetts Institute of Technology, it was arranged for the District to do the field work and the Institute to make the laboratory tests and experiments.

NOTE.—The paper by Glennon Gilboy, Jun. Am. Soc. C. E., was presented at the meeting of the Structural Division, Boston, Mass., October 10, 1929, and published in October, 1931, *Proceedings*. Discussion of the paper has appeared in *Proceedings*, as follows: December, 1931, by Messrs. J. C. Meem, and H. deB. Parsons; and January, 1932, by Messrs. Jacob Feld and John R. Jahn.

[30] Chf. Engr. and Gen. Mgr., Miami Conservancy Dist., Dayton, Ohio.

[30a] Received by the Secretary December 14, 1931.

[31] *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), pp. 1195-1197.

Two general methods of obtaining samples were available. One was to sink a shaft large enough for men to work in and obtain samples, and in which the materials could be inspected in their undisturbed condition. The other method was to secure the samples from some type of drill hole or boring. In order to obtain the relative value of the two methods, it was determined to use both. The shaft, of course, would be much more expensive than the boring, but it was fairly certain that good samples could be obtained by this method, whereas some uncertainty existed as to the value of those from a drill hole or boring. The primary consideration in either case was to obtain the samples as nearly as possible in an undisturbed condition.

During the construction of these dams Goldbeck pressure cells[32] had been installed at Germantown and Taylorsville. The most satisfactory results from these cells were obtained at Taylorsville, and it was at first thought desirable to obtain the samples from this dam, but because of the paved roadway and heavy traffic conditions a vertical shaft appeared impracticable there, while a sloping shaft or drift would have been more expensive and a lesser depth of core would have been penetrated by it. On account of its greater height and the accessibility of the core, the Germantown Dam was finally selected.

The shaft was located about 12 ft. west of the center line of the dam at a point where its height was approximately 110 ft. A shaft on the center line would have been more desirable, but was impracticable on account of its obstruction to traffic over the dam which is traversed by a public road.

It was decided to sink a vertical shaft with a circular concrete casing, with walls 5 in. thick and 5 ft. in internal diameter. A sheet-metal form somewhat similar to those used for monolithic concrete silos, was provided. The casing was cast at the ground level in sections about 3 ft. high. As the excavation proceeded the concrete sank under its own weight and new sections were cast in place on top of the old ones as the sinking proceeded. As there is no true core in the upper 15 or 20 ft. of the dam and the shaft was located to one side of the center line, no core material was encountered until a depth of 33 ft. was reached. The first set of samples was secured at this elevation, Professor Gilboy being present when they were taken.

The excavating for the shaft was done by hand by one man, and the material elevated with a hand-hoist and derrick (see Fig. 14), the boom being placed so that in a lowered position the bucket could be dumped into a wagon beyond the shaft. A small shed to house materials, tools, and samples was constructed to act as a counterweight for the boom. Being placed to one side of the center line more layers of fine gravel and sand were encountered than would have been the case had the shaft been directly on the center line of the dam. This led to the unexpected difficulty of handling a considerable quantity of water contained in these layers, which entered the shaft beginning at a point within 30 ft. of the top of the dam and far above the maximum water level ever recorded in the reservoir. A hand-pump was installed but as the excavation proceeded pumping by hand became too tedious and a small gas engine and pump-jack were provided for this work.

[32] *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 1188.

The water encountered had undoubtedly remained from the construction operations, with some possible addition due to the rainfall on the dam. It is retained by layers of core material which project outward from the main core between the layers of gravel, all the layers sloping downward toward the center of the dam. The retention of this water is evidence of the imperviousness of the core material.

The tools for obtaining samples are shown in Fig. 15. They were designed by Professor Gilboy under the direction of Charles Terzaghi, M. Am. Soc. C. E. At (*a*) is shown a seamless steel sampling tube, 4 in. in diameter, and 12 in. long, with one end turned down to a knife-edge, and this edge burnished in 0.001 in. to give clearance to the sample. The tube, which had been wiped with an oiled rag before using in order to reduce friction on the inside, was forced into the core material by hand, the material around it being removed with a trowel as fast as the tube penetrated. As soon as the tube was filled it was carefully removed and the surplus material at the ends was cut off by means of a fine wire in a hack-saw frame. Sheet metal caps (*b*) were then placed over the ends and the sample was brought to the surface. Samples were taken in pairs at each location, one vertically and the other horizontally, at right angles to the axis of the dam.

On the work table were two circular iron bases, $\frac{3}{4}$ in. by $4\frac{1}{2}$ in. (*c*). When a sample was brought to the surface to be prepared for shipment, the iron base was covered with a thin layer of melted paraffin and the sample set vertically on it after the caps had been removed. The steel tubing was then slipped from the sample. If it did not come off easily, a wooden disk slightly smaller in diameter than the tube, was placed on top of the sample and held down while the tubing was pulled up off the sample. A cylinder (*d*) of 24-gauge galvanized iron, $4\frac{1}{2}$ in. in diameter, and $13\frac{1}{2}$ in. long, was then placed over the sample, the bottom sealed with modeling clay, and the space around and above the sample filled with melted paraffin. When the paraffin had hardened the cylinder was reversed, heat being applied to the iron disk if necessary to loosen it, and the other end poured full of the paraffin. The entire sample was thus enclosed in paraffin, the structure of the sample and its water content being well preserved until it arrived at the laboratory. The tube (*e*) was used for obtaining samples from the drill hole.

When the work began it was the intention to obtain a pair of samples every 10 ft. This was found to be impracticable and only four good sets of samples were secured. The first 30 ft. was not in core material, and the shaft was sunk to a depth of only 80 ft. At a depth of 40 or 50 ft. the casing separated at one of the joints and side-slip occurred as it continued to sink. It was necessary to install some timber braces to prevent this. At this time the sinking process was very slow, and it became necessary to weight the casing and to force it down with jacks. At a depth of about 60 ft. it appeared impracticable to sink it any farther because it moved out of alignment and some cracks occurred. At this point a new casing was started inside the first one, the concrete being taken down in buckets. This was sunk to an additional depth of 20 ft. After a depth of 40 or 50 ft. had been reached it was necessary to supply air to the workman at the bottom of the

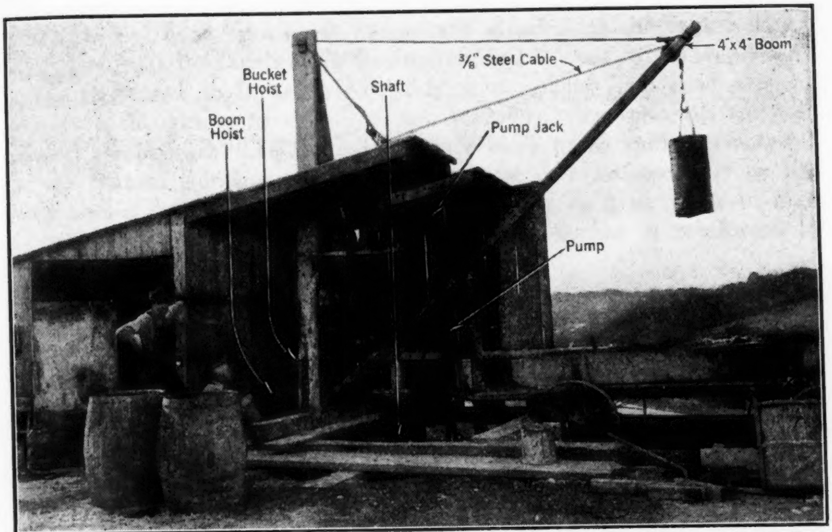

FIG. 14.—ARRANGEMENT AT SHAFT HEAD, GERMANTOWN DAM, FOR SINKING TEST PIT.

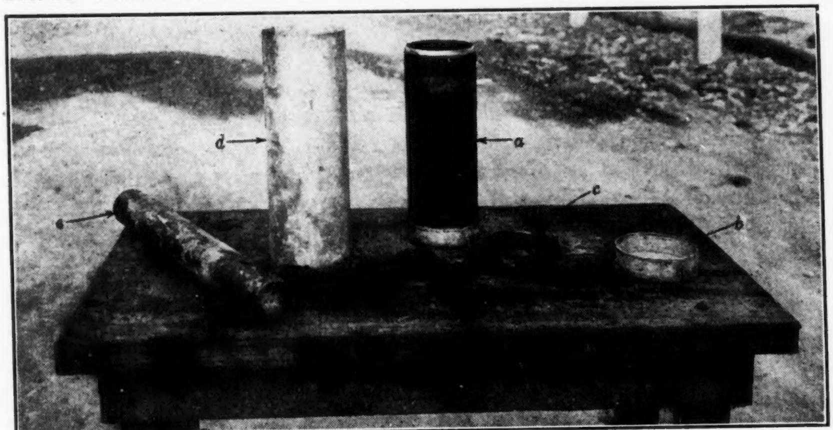

FIG. 15.—APPARATUS FOR TAKING SAMPLES OF CORES, GERMANTOWN DAM.

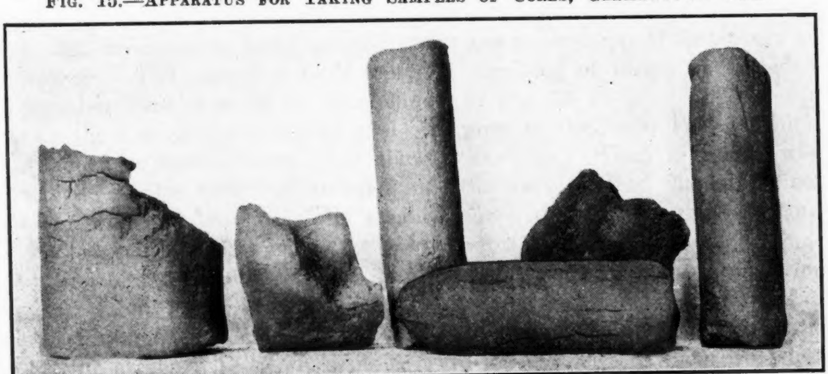

FIG. 16.—CORE SAMPLES TAKEN FROM GERMANTOWN DAM IN 1927.

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shaft. This was done by a small compressor attached to the engine which ran the pump. At the depth of 80 ft. progress had become rather slow. The shaft also appeared to be somewhat dangerous. The four good sets of samples were thought to be sufficient and, therefore, the work was stopped.

The writer believes that if another shaft were to be sunk in material similar to that at Germantown, it could be done very satisfactorily by the methods herein described, but that the walls should be 8 in. thick to give them additional weight for sinking and that they should be reinforced to prevent any separation at the construction joints.

In order to determine whether or not any of the porous strata encountered, extended across the center line of the dam, 4-in., horizontal holes were bored at right angles to the center line in the two heaviest layers of this kind. In both cases, solid core material was encountered within a few feet of the shaft.

The samples secured were approximately of the nature of rather stiff putty and would stand on end unsupported, when removed from the sampling tubes. Pieces of the excavated core material would stand a great deal of handling without losing their shape, and considerable pressure was necessary to indent them with the fingers. Professor Gilboy states that the laboratory experiments show the core material to be 25% consolidated and explains that this does not mean that it is soft or yielding, but that it is quite resistant due to its relatively high angle of internal friction. This point should be emphasized. The simple statement that the core is 25% consolidated may be misleading to some unless a comparison is made with other materials, the qualities of which are better known. The writer has been informed that Boston blue clay, which is considered a satisfactory material for building foundations, is only 10% consolidated by the same standards. The future rate of consolidation of the Germantown core is necessarily somewhat uncertain, but from a practical standpoint this is unimportant. The quantity of water encountered in sinking the shaft shows that there is no danger of it becoming too dry while its present state of consolidation makes it entirely satisfactory for core purposes.

The drill hole was located on the opposite side of the center line of the dam within about 25 ft. of the shaft. An ordinary well-drilling outfit was used, the pipe being $5\frac{5}{8}$ in. in diameter. The work was done by a competent well driller, the samples being secured under the supervision of employees of the District. The sampling tools provided consisted of pieces of threaded 2-in. seamless steel pipe, 24 in. long, shown in Fig. 15 at (*e*). The rod for lowering these samplers consisted of $1\frac{1}{4}$-in. pipe, to the lower end of which was attached a piece of 2-in. pipe about 2 ft. long. When a sample was desired, the sample tube was screwed into the lower end of the 2-in. pipe. After the well had been carefully cleaned out with a dipper the sampling tool was lowered and driven into the core about 3 ft. This insured the complete filling of the sampling tube. The samples remained in these tubes for shipment, about $\frac{3}{4}$ in. of the sample being removed from each end and replaced with melted paraffin. The drill hole was driven to the base of the dam. Nine attempts to secure samples were made. Two of them were lost

by slipping from the tube, but the others appeared to be very satisfactory. In commenting on the results of the laboratory tests Professor Gilboy has stated,[33] that he feels convinced that, for homogeneous material, a good drill-hole sample will furnish results as reliable and consistent as the undisturbed sample from the shaft. In Fig. 16, the cylinders with the larger diameters are samples from the shaft; the others are from the drill hole.

In addition to the shaft and drill-hole samples of core material, a number of samples were taken from the borrow-pits at the Germantown Dam and sent to the laboratory to be analyzed and tested. While these were as nearly representative of the materials used for the dam as could be secured, the writer does not believe that they can be truly representative because they were necessarily obtained from the edges of the borrow-pits and may vary somewhat from the average of the materials obtained from these pits.

At the laboratory, mechanical analyses were made of the core samples. They were also tested for permeability, compression, compressive strength, angle of internal friction, water content, and specific gravity. It was necessary to design a new permeameter at the laboratory and other pieces of apparatus were modified to better meet the requirements.

The mechanical analyses were consistent with those made during the construction period. The average rate of permeability, as expressed by Professor Gilboy, was approximately 100×10^{-7} cm. per min., a very low rate.

Tests for compression and compressive strength showed that the material possessed a high degree of stability. The friction angle averaged about 27 degrees. The water content for a number of samples, expressed in percentage of dry or solid matter, averaged approximately 25 per cent. The specific gravity was found to be 2.67 for the solids and 2 for the mass. This gives a weight of 125 lb. per cu. ft. for the undisturbed core.

As a result of this work the following conclusions may be drawn:

1.—The samples from the drill hole were fully as satisfactory as those from the shaft, and were obtained with much less expense.

2.—The mechanical analyses showed the core material to be well graded from fine sand to clay in such proportions as to make a combination ideally adapted for a hydraulic-fill core.

3.—The water content does not differ materially from that found at Englewood and Lockington at the end of the construction period. While the Germantown Dam was not tested for water at that time, the mechanical analyses show the material to be quite similar. This would seem to indicate that there has been little change in the cores during the seven years between 1920 and 1927 and that they will change but little in the future.

It is hoped that the tests may be repeated at intervals of five or ten years, and that similar tests will be made on other dams.

Those in charge of the Soils Laboratory at the Massachusetts Institute of Technology are to be highly commended for the work they have done in developing the laboratory methods described in Professor Gilboy's paper.

[33] In his thesis for his Doctor of Science degree.

A. A. EREMIN,[34] ASSOC. M. AM. SOC. C. E. (by letter).[34a]—Present interpretation of soil behavior under sustained loads is erroneous and unsafe. The soil problem is complicated, due to the facts that no two foundation materials can be found alike and that it is difficult to obtain test samples under natural conditions.

Professor Gilboy has outlined the characteristic factors of soil mechanics concerning which a study is desirable in order to derive reliable rules for interpreting the physical properties of foundation materials. In the study of soil attention should be given to the effect of colloids. The chemical analysis of colloidal materials is laborious and expensive. Furthermore, the results of the analysis obtained with the present state of knowledge often conflict with actual practice. However, to neglect the colloidal effect would introduce considerable error in the study of permeability and cohesion of soil, which are the most important factors in soil mechanics.

Tensile strength of loams and clays increases with the increased colloidal content, as has been shown[35] by John H. Griffith, M. Am. Soc. C. E. Therefore, the angle of internal friction of the test cylinder, Fig. 9, may also vary due to the binding effect of the colloidal particles between soil grains.

Likewise, the permeability of soil varies, due to expansion and contraction of the colloidal materials and to its reactions with the chemicals in the ground. From research on soil mechanics for hydraulic dam cores, Mr. F. E. Hance has stated that soil chemically treated may become from 200 to 1 000% more impermeable than the same material when neutral.[36]

In the study of foundation materials an investigation of the bearing on piles in compressible soils should be included. In the future, estimation of the bearing capacity of piles will probably be based on the physical properties of the foundation material. Moreover, the penetration curve of piles may give useful information about the character of the soil.

The Engineering Profession should encourage the study of soil mechanics because the advancement of the science of foundation materials will bring improvement in the economical consideration related to structural designs.

F. N. MENEFEE,[37] M. AM. SOC. C. E. (by letter).[37a]—The subdivision of this study into (1), soil physics, and (2), soil engineering, is good. It is analogous to the division of other studies of an engineering nature. The very refined, detailed, and sometimes little used, although valuable, properties of a material correctly belong in a classification by themselves, for a larger part of the engineer's professional knowledge is based upon scientific studies which at the time they were made had no direct application. In the case at hand Charles Terzaghi, M. Am. Soc. C. E., and the author have aptly called that part of their study soil physics, and that which follows, soil engineeering.

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[34] Asst. Designing Engr., Bridge Dept., State Highway Comm., Sacramento, Calif.

[34a] Received by the Secretary December 7, 1931.

[35] "Physical Properties of Earth," by John H. Griffith, M. Am. Soc. C. E.

[36] *Engineering News-Record*, October 3, 1929, p. 542.

[37] Prof., Eng. Mechanics, Univ. of Michigan, Ann Arbor, Mich.

[37a] Received by the Secretary December 15, 1931.

Due to studies by Duff A. Abrams,[38] M. Am. Soc. C. E., and the late William B. Fuller,[39], M. Am. Soc. C. E., on concrete aggregates, American engineers in general have adopted the practice of plotting their mechanical analysis curves with the smaller diameter at the origin, thus giving a curve of positive rather than of negative slope, as does the author. There is no question of right or wrong to either method, but standardization of method in presenting data does assist interpretation and facilitates the use of an idea.

Since Mr. Fuller's ideal curve gave the densest mixture of sand and gravel it probably would assist in determining the permeability as applied to soils because, aside from the possible influence of colloids and swelling tendencies of, or chemical changes with addition of, moisture, the hydrostatic head lost in water flowing through soil is inversely proportional to the diameter of the voids. To state it in another way, the velocity of water through soil increases or decreases with increase or decrease in diameter of the inter-granular space. An ideal gradation from fine to coarse fills up the interstices and reduces the permeability.

Atterberg's choice of terms—"liquid limit," "plastic limit," and "shrinkage limit"—seemed very good to the writer, but the methods of determining them were subject to at least some personal equation. The methods described by the author seem to be an improvement.

The writer is not familiar with Mohr's theory as outlined by Professor Gilboy, but finds that Mohr's circle method of finding planes of maximum shear on a cylinder, such as shown in Fig. 9, gives a 45° plane the same as ordinary mechanics. However, it is known that concrete and earth do not always fail from shear on 45° planes when subject to compression.

The resistance to shear and the total cohesion or tensile resistance in soil may be greatly increased by mechanical working or by exciting high

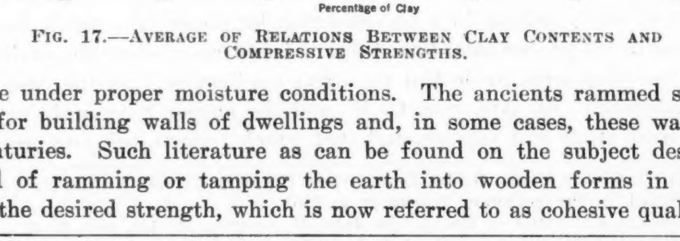

FIG. 17.—AVERAGE OF RELATIONS BETWEEN CLAY CONTENTS AND COMPRESSIVE STRENGTHS.

pressure under proper moisture conditions. The ancients rammed soils into forms for building walls of dwellings and, in some cases, these walls stood for centuries. Such literature as can be found on the subject describes a method of ramming or tamping the earth into wooden forms in order to attain the desired strength, which is now referred to as cohesive quality.

[38] *Bulletins*, Structural Materials Research Laboratory, Lewis Inst., Chicago, Ill.

[39] "The Laws of Proportioning Concrete," *Transactions*, Am. Soc. C. E., Vol. LIX (1907), p. 67.

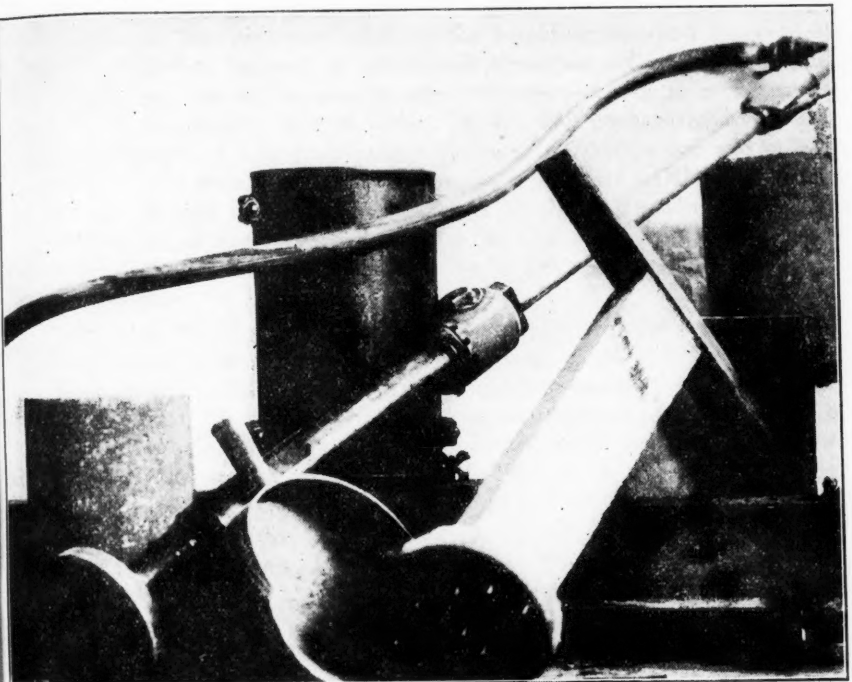

FIG. 18.—APPARATUS USED FOR MAKING SPECIMENS.

FIG. 19.—CHARACTERISTIC FAILURES.

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The writer has performed a series of experiments on mixtures of clay and sand rammed into cast-iron cylinders for moulding standard concrete specimens. Fifty-four bags of inorganic soil were first collected for use in the investigation. As far as possible the soils were chosen to represent every subsoil in the locality of Ann Arbor, Mich. The compositions varied from the one extreme of a very heavy clay binder mixed with a fine and pulverized sand, to one in which the sand predominated similarly. Others had a coarse aggregate bonded by varying percentages of clay. Knowing the sand and clay analysis of each sample, in order to obtain the composition of soils between those represented by the samples, it was sometimes necessary to mix two or more.

The first step in the forming of the test cylinders, and a step essential to uniformity of product, was a thorough mixing of the soil to be tested with sufficient water to fulfill the rough requirements described[40] by Messrs. Edward W. Coffin and H. B. Humphrey. A sample squeezed tightly in the hand should adhere in the form of a ball, but will break into small pieces when dropped to the ground from the height of the waist line. This property is essentially governed by the clay content of the sample, but the writer found that over a large range of soils, varying in clay content, a tempering with water to fulfill this test gave a mixture that could be best handled with the air-hammer.

These moulds were clamped to a metal base which, in turn, was screwed to a heavy wooden section to dampen the jarring during the tamping. Enough of the sample was placed in the mould to equal a depth of about 5 in. and this was hammered until it sounded and felt solid. The surface was then scarified and the operations repeated until the mould was filled to a depth of 9 or 10 in.; above this height it was practically impossible to keep the rammer within the mould. This left the surface uneven, so that before removing the material from the mould a load of 4 000 lb. was applied to the surface of all samples alike. While this load was too small to change the density of the material appreciably it did tend to present a smooth surface for later testing. The specimens were then stored away and seasoned, most of them for twelve months.

The results of the tests are shown in Fig. 17. It will be noted that a value of 400 lb. per sq. in. was attained in one instance. At a later date a cylinder of about the same clay and sand content was attained which withstood nearly 500 lb. per sq. in. of compression. The apparatus used for making specimens is shown in Fig. 18.

The type of failure was very much the same as that found in concrete. The cone to the left in Fig. 19 is of concrete, whereas the three specimens to the right are of rammed earth. It is believed that these failures depart from the 45° plane because they are due to tension more than to diagonal shear.

The author's Fig. 9(*a*) shows a barrelling effect of the clay cylinder which in itself indicates that the diameter increases between the top and bottom, introducing tension.

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[40] Handbook on "Building Walls with Rammed Earth."

Fig. 20 represents a clay cylinder subject to compression; P is the total load, and f is the frictional resistance tending to keep the ends from spreading or increasing in diameter. This friction is a maximum at the periphery

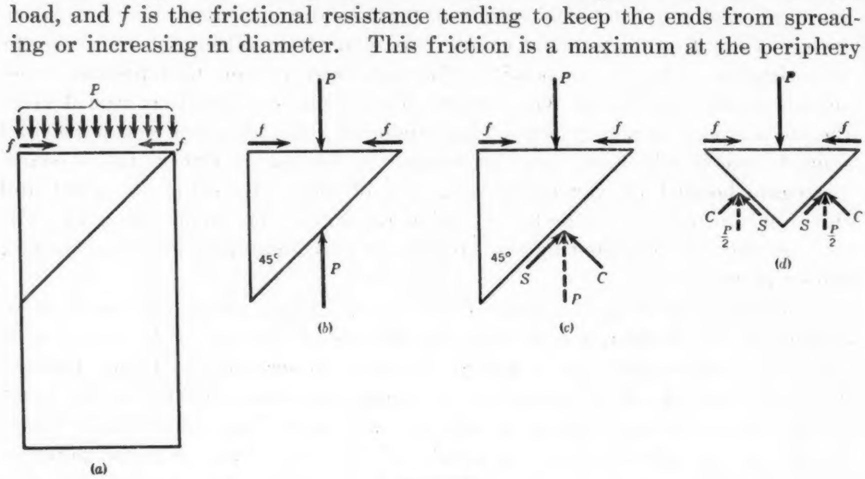


FIG. 20.

and diminishes to zero at the center or axis of the cylinder. If f could be eliminated the maximum shear would be,

$$S = \frac{1}{2}\,\frac{P}{A} \qquad (6)$$

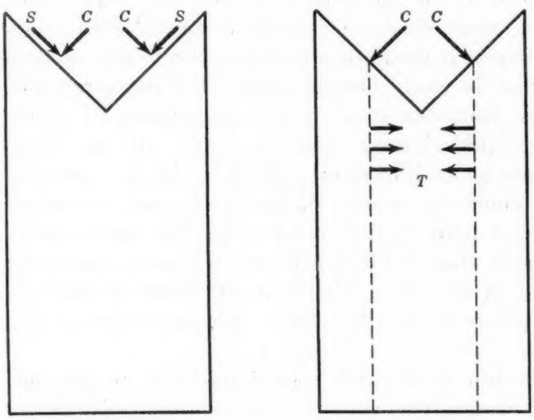
in which, A is the cross-sectional area, and would be on planes 45° with the upper and lower faces. (Fig. 20(a), Fig. 20(b), and Fig. 20(c)). With f always existing to a certain extent the plane of maximum shear will be less than 45° with the upper and lower faces, or the cone will be flatter than shown but actual tests show that the failures in earth cylinders and concrete as well, are along planes of greater slope than 45°, as indicated in Fig. 19.

From a purely static consideration, the shearing and comprehensive forces on the slant faces of the cone neutralize each other and the resultant splitting tendency is nil; but when the load is applied the forces actually produce deformation, and the cone settles into the cup (Fig. 21(a)), through shearing deformation. Thus, the values of S are reduced, the values of C increase, and spreading action takes place introducing tension in concentric cylinders from top to bottom (Fig. 21(b)).

The illustration is made clearer by assuming a highly polished cone placed into the correspondingly polished cup (Fig. 21(b)). The values of S disappear, and the values of C increase until the sum of their vertical components is equal to P, their horizontal components pushing outward, increasing the diameter of the cylinder. Although there are no external forces pulling the sides of the cylinder outward the internal horizontal forces pushing outward radially from the axis produce diametral strain and corresponding stress.

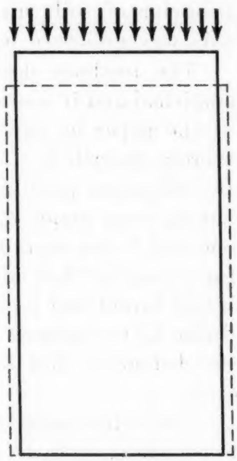
Unfortunately, the writer does not know Poisson's ratio for earth, but there is a direct analogy between the qualitative analyses of the behavior of solid earths and concrete.

Knowing Poisson's ratio for concrete to be about 0.12 and taking $E = 3\,000\,000$, a compression of 4 000 lb. per sq. in. will produce $\frac{4000}{3\,000\,000} = 0.00133$ in. per in. of strain or shortening (Fig. 22). This will be accompanied by a diametral elongational strain of $0.12 \times 0.00133 = 0.00016$ in.



(a) (b)

FIG. 21.



(c)

FIG. 22.

per in. The corresponding tensile stress would be $3\,000\,000 \times 0.00016$, or 480 lb. per sq. in., which is enough to cause a tension failure that is becoming known as "longitudinal split." The failures of earth cylinders shown in Fig. 19 indicate a similar behavior.

E. G. WALKER,[41] M. AM. SOC. C. E. (by letter).[41a]—This paper is of value to engineers interested in the practical application of the mechanics of soils because it is a concise account of the lines along which research in this comparatively new branch of engineering science is being pursued. It would appear from the paper that the laboratory investigations into the physical properties of soils are being developed with a view to obtaining data on four general properties, namely: (1) Their mechanical analysis; (2) their "limits of consistency"; (3) their permeability; and, (4) their internal friction and cohesion.

As far as these properties are real physical values, a knowledge of their variations relative to one another and to external phenomena may be expected to be of considerable use to the engineer in the future in assisting him to deal with the many practical problems which arise from day to day in civil engineering practice. At present, most of these problems can only be solved, in the light of previous experience.

[41] Maxted & Knott, London, England.

[41a] Received by the Secretary December 21, 1931.

The subject is undoubtedly one of the greatest complexity and—no matter how successful laboratory research ultimately may be in connecting the various physical properties of individual soils with their behavior in Nature—the practical application of the results is almost certain to be fraught with uncertainties. At the same time, this is no argument against the pursuit of laboratory research, because this should enable engineers to predict the behavior of soils under the ideal conditions of experimental work, and in this way guidance may be given to the application of practical experience.

The methods described for measuring consistency certainly seem very empirical and it would be of interest if the author would elaborate this section of the paper by explaining what is really obtained from the measurements therein described. The change from the state of wet consistency in which the clay acts practically as a liquid down to that in which, all the water having been removed, it may act as an imperfectly elastic solid, is a gradual one and is not marked by any complete changes of properties, such as occur, for example, when an increase of temperature converts a solid, successively, into a liquid and a gas. It seems undesirable, therefore, to attempt to define limits in the manner suggested in the paper, since this straightway leads to the deduction that there are points at which definite changes of properties exist.

The writer suggests that the line of research should rather be to attempt to correlate, with the variation of moisture content, some representative property of the clay, such as the force necessary to produce some agreed distortion of the sample. If a detailed study of the phenomena should show points to exist at which changes of properties become apparent, a more scientifically defined limit could be designated.

The writer would like some further explanation of Equation (1). The quantity, $\dfrac{W_o - W_o}{W_o}$, is the change in weight due to drying divided by the weight of the dry sample and is, therefore, the ratio of the weight of water to clay in the wet sample. The quantity, $\dfrac{V_1 - V_o}{W_o}$, is the ratio of the shrinkage to the weight of the dry sample and is, therefore, the reciprocal of the apparent change of density of the clay. Hence, this equation requires that a measure of bulkiness, which cannot be dissociated from units of weight and volume, be subtracted from an abstract ratio in order to obtain a value which is called a "shrinkage limit." The task of defining limits becomes doubly difficult when it is proposed to measure them in so obscure and empirical a manner.

The portable capillary permeameter shown in Fig. 5 would appear to be a good type of apparatus for approximate field determinations when the more exact apparatus shown in Fig. 4 would not be available. The author states that the coefficient, *F*, in the formula used for reducing the measurements made with the horizontal capillary permeameter, may be taken as 100 000, with a range of deviation of ± 50 per cent. Equation (5) reduces to

$k = \frac{x^4}{t^2} \times \frac{e}{1 + e} \times \frac{1}{F}$; that is, for any given set of observations the permeability varies as $\frac{1}{F}$. Hence, if F may vary within the stated limits the actual permeability may vary from two-thirds to twice that calculated by assuming a mean value, $F = 100\,000$. Therefore, unless calibration of the instrument against the standard type of permeameter can reduce the range of possible variation of F, the capillary type does not appear to be of much practical value.

Research with the consolidation apparatus, in which the sample is constrained laterally, together with mechanical tests in compression and shear on the lines described under "Internal Friction and Cohesion," should lead to results of considerable practical value. It is hoped that this part of the work of the laboratory will be developed to the fullest extent in combination with the field testing of foundation soils described under "Foundations."

The four classes of engineering researches described in the second part of the paper foreshadow great possibilities of adding to the knowledge, facts that will enable the design of civil engineering works to be placed on a more satisfactory basis.

As far as the writer is aware, no organized research on the pressures on retaining walls has ever been attempted before with so large an apparatus as that shown in Fig. 13. Many experiments have been made on earth pressures during the last hundred years and more, but the great majority have been useless as far as the direct practical application of the results to engineering design is concerned, solely because the scale upon which the experiments have been conducted, has been too small. It has not been recognized sufficiently in the past that there must be a distinct scale effect in experiments of this nature. Whereas a model wall quite conceivably may be constructed to a small scale to represent the original in dimension, weight, distribution of material, etc., to a close degree of exactness, it is impossible, proportionately, to scale down the soil which presses against it, or to represent adequately the effects of consolidation or loosening of the soil resultant from actual practical works of construction. The only hope of obtaining better design data than are now available is to carry out experiments on a large scale. The writer has more sanguine hopes of the success of the large apparatus described in the paper, therefore, than he has had of previous researches of this nature.

There is one detail, however, which the writer ventures to criticize. The bin in which the earth is to be retained is 14 ft. square and 10 ft. high. That is, the ratio of width to height of the measuring panel is only 1 to 4. With so low a ratio, unless special precautions are taken, the measurements are likely to be distorted considerably by end effects. The introduction of watertight joints between the movable panel and the side walls, when experiments on saturated soils are to be undertaken, will only accentuate this difficulty. As no mention of this point is made in the paper, it would be of interest to learn how it is proposed to deal with it.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

ENGINEERING FEATURES OF THE ILLINOIS WATERWAY

Discussion

BY MESSRS. A. W. SARGENT, AND E. G. WALKER

A. W. SARGENT,[12] M. AM. SOC. C. E. (by letter).[12a]—Some new and interesting features in lock design have been brought out in this paper. The use of the Venturi principle in the design of lock culverts has resulted in a saving in the cost of valves and valve-operating machinery with little loss in efficiency. The valves, which apparently are designed to be lowered by their dead weight, without strut or stem to force them to the closed position, would operate satisfactorily in quiet water and under a small head, but it is the writer's opinion that some mechanism should have been provided to force them to their seats under a head equal to the maximum lift of the lock. It will be interesting to know whether in actual practice the valve-operating mechanism, which is quite simple and inexpensive compared with many other types now in use, has proved entirely satisfactory; and, also, it would be interesting to know the maximum head under which a valve can be lowered to its seat.

The design of the gates, with air chambers in the lower part, single skin on the upper part on the tension or down-stream side, and buckle-plates for sheathing, has no doubt resulted in economy, but the additional protection afforded by a complete double skin would be beneficial. It would prevent the accumulation of driftage on the girders and the painted surfaces would be better protected. The interior surfaces of the double-skin gates of the Lake Washington Ship Canal, which is between salt water and fresh water, require little attention. A coating of bituminous solution and enamel applied about 1916 is still (1931) in excellent condition, while the exterior surfaces have required extensive repairs annually when the lock is unwatered for general overhauling of all under-water equipment.

NOTE.—This paper by Walter M. Smith, M. Am. Soc. C. E., was presented at the meeting of the Waterways Division, Milwaukee, Wis., July 11, 1929, and published in October, 1931, *Proceedings*. This discussion as published in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

[12] Civ. Engr., U. S. Engr. Office, Govt. Locks, Seattle, Wash.

[12a] Received by the Secretary December 11, 1931.

The design of the part in contact with the miter-sill is new, and the writer believes it will be uniformly satisfactory when the strut-arm method of operating the gates is used (Fig. 9). This method does not allow the leaf to swing beyond the proper mitering position. The motion is eccentric at the heel and does not contact with the quoin-post except when mitered (Fig. 4). Where gates are operated by wire cables a stop should be provided to prevent a leaf from swinging too far and injuring the sealing strip as well as stressing the hinge-pins and anchorage. The writer had this in mind when he designed the water-seals for the lower guard-gates of the Lake Washington Ship Canal.[13]

To insure the successful operation of an emergency dam when an emergency arises, all parts should be easily accessible for inspection and repair. A structure of this kind which is continually submerged is not considered the best type. This condition could possibly have been avoided by placing the guard-gate up stream from the emergency dam. All parts of the dam would then be accessible after unwatering the lock. The spar bull-wheel type of gate-operating machinery which is now generally used for mitering lock-gates, has proved very satisfactory.

The Lockport Lock, which has a lift of 40 ft., is one of the highest in the United States. The Wilson Dam Lock has a lift of 45.5 ft., and the Keokuk Lock, one of 41.0 ft. The locks of the Welland Ship Canal, in Canada, have a maximum lift of 46.5 ft. The difficulty of handling vessels in high-lift locks, while the water is being raised and lowered, is quite apparent. At the Welland Ship Canal mooring passages are provided in the lock-walls to facilitate the mooring of vessels at lower pool levels.[14] This method possibly has some drawbacks as the passages no doubt become slippery making it dangerous for the men handling the lines.

The writer believes that it would be practicable to equip high-lift locks with movable mooring devices which could be held in vertical recesses in the lock-walls, and raised or lowered with the water, either mechanically or by floats. A mechanism equipped with roller-bearings would facilitate the raising or lowering when a ship's lines are taut. A ship entering a lock which has been equipped with these devices, would have the eye on the mooring lines attached to two or more as desired and this should be secured to the ship's cleats or windlass. Lengthening and shortening of lines, as is usual when lowering or raising a vessel, would not be necessary. The ship's crew would do the principal work now required of the linesmen on the lock-wall, and the safety of the vessel in the lock would be dependent upon the vessel's crew and its own lines. With workable mooring devices which would be held a certain distance above the water in a lock while the water is being raised or lowered, the operation of locks of extreme lift would be feasible and much of the difficulty of providing lockage facilities at high dams would be overcome, and the time of passage between pools would be reduced.

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[13] *Transactions*, Am. Soc. C. E., Vol. 92 (1928), p. 1008.

[14] *Proceedings*, Am. Soc. C. E., October, 1929, Papers and Discussions, p. 2066.

E. G. WALKER,[15] M. AM. SOC. C. E. (by letter).[15a]—The arrangement of the sluices on the culverts of the locks along the Illinois Waterway is certainly unusual, and it would be interesting to learn whether the advantages claimed for the reduction of the size of the valve, relative to the cross-section of the culvert, are fully justified in practice. In order to obtain efficiency, the principle of the convergent and divergent mouthpiece which was used by the late Clemens Herschel, Past-President and Hon. M. Am. Soc. C. E., in the development of the Venturi meter, presumes a gradual convergence to the smallest cross-section and subsequent divergence of the stream line without varying the general shape of the stream, except possibly by such gradual changes of curvature as will prevent the formation of eddies and turbulence.

In the design which the author describes, a circular culvert, 12 ft. in diameter, is reduced at the valve face to an opening 9 ft. square. In practice, any transition from the circular to the square must introduce a certain amount of eddying, however gradual it may be, and, therefore, the full benefit of the contraction of sections cannot be obtained. The type of sluice-valve described by L. D. Cornish, M. Am. Soc. C. E., as being used on the Illinois Waterway,[16] could not readily be adapted to suit a circular opening, and this accounts for the change in cross-section of the water passage. The principal benefit that is obtained by using a smaller sluice-valve is that the cost of the valve is thereby reduced. It does not seem likely that there can be much practical value in the theoretical hydrodynamical advantage of increased rate of flow, when the practical circumstances of the problem are considered. The author calculates a theoretical advantage of only 25 sec., and it seems doubtful whether even this could be fully justified if a practical comparison could be made between the larger and smaller valves.

The proportion, 1 to 4, of rise to span for the lock-gates seems high. It is stated that it was adopted as the result of estimates based on actual designs. Is it to be assumed that these were designs prepared in connection with the job, or is it the author's generalization of an examination of previous practice? If the latter it would seem that he has examined only the more abnormal cases. The writer knows of many instances of mitered gates in which the rise is as little as one-sixth of the width between the centers of the quoin-posts, and he believes that although cases can be found, especially on small locks, of rises of one-fourth and even one-third of the width, an average would probably be considerably less than one-fourth.

It would be of interest if the author would enlarge a little upon his reasons for stating, as he does under "Miter Lock-Gates," that mitering lock-gates should be of the parallel type. It is common practice in America for this to be so, but in many other places the cylindical-backed gate has been found to be suitable, and there are definite features in favor of it. The parallel gate has the advantage of taking up considerably less space in the lock-wall when open, and may enable a reduction to be made in the over-all width of

[15] Maxted & Knott, London, England.

[15a] Received by the Secretary December 21, 1931.

[16] Manuals of Engineering Practice, Am. Soc. C. E., No. 3, on Lock Valves, Appendix 3, p. 17.

the lock between the backs of the walls of the gate recesses (usually the widest part of the structure). In many instances, particularly when two locks or entrances have to be constructed side by side in a limited width, this may be a great advantage in enabling the works to be arranged so as to give the most economical utilization of the available space. On the other hand, the cylindrical-backed gate does not present any difficulties of construction; the plates have only to be rolled to a relatively simple curvature in one plane and this applies to all the structural details involved. Therefore, it does not present any construction difficulties as compared with the parallel gate. At the same time, it has the advantage that the line of thrust can usually be kept well inside the thickness of the gate at all points, and thus enable better distribution of metal to be arranged with smaller variation of stress from point to point of the structure. It is noted that in Fig. 6 (*a*), the value of j (the distance of the line of thrust from the chord of the gate) reaches a maximum of 8.226 ft., as compared with a gate width of 7 ft.

The writer was interested to learn of the use of buckle-plates on the gates, a feature which would not be feasible with cylindrical-backed gates. This appears to be somewhat of a new departure in lock-gate construction and it is of value to learn that a saving of cost was effected thereby. If such saving can be effected in the general case it is a strong feature in favor of the use of parallel gates.

The statement in the paragraph introducing Fig. 5 is peculiar. It seems to the writer that in all lock-gates there is a self-evident benefit in endeavoring to balance as much of the weight of the gate by buoyancy as is suitable to the average circumstances under which the gate is to operate. This applies whether the gate is small or large. If the weight of the gate can be balanced by flotation there must be a self-evident advantage in equalizing the distribution of pressure on quoins, miters, and sills, and, therefore, of enabling more water-tight joints to be made and maintained. All the gates on the Illinois Waterway are certainly large enough to have made the provision of buoyancy chambers worth while and it seems remarkable, therefore, that any attempt to design gates without buoyancy chambers should have been made at all.

The writer would suggest the desirability of adding to the paper the detail of the miter-post, similar to the clear detail of the quoin-post shown in Fig. 4. He suggests that this, in company with the other details of the gates given in the other diagrams in the paper, would complete the author's clear description of the gates as a whole.

The design of the pintle, as shown in Fig. 5, while it is in accordance with American practice, seems to be rather more complicated than is necessary. With properly supported gates in which, under most of the conditions of operation, the weight is balanced by the buoyancy of the air chambers, there should not be a large load upon the pintle and, therefore, a plain steel casting with a corresponding plain cast-steel cup on the heel of the gate, should be sufficient and would appear to be a cheaper solution.

The design of the sill on the miter-gate is an interesting attempt to overcome the disadvantages inherent upon the older methods of attempting to

make a tight joint by means of timber or metal clapping sills. Provided no obstruction gets between the lock-sill and the spring-plate, there should be every possibility of making a tight joint, if the spring-plate has been properly adjusted in the first place. There are, however, one or two points upon which the writer would like information. It would be interesting to learn the amount of deflection allowed in the design. It is possible that small obstructions might easily cause inordinate leakage. With a solid clapping sill the introduction of such obstruction would be sufficient to prevent the gate from closing properly and so most likely the existence of the obstruction would be made known. With the spring-plate, however, it seems quite conceivable that the obstruction might force the plate back, but yet not be sufficient to prevent the gate from mitering, and in this way a permanent opening would be formed along the sill of the gate after the lock had ostensibly been closed, and considerable leakage would thus result. There is also the possibility of damage to the plate by obstructions, such that the damage could only be discovered by under-water inspection and yet not be of such a nature as would prevent the gates closing properly.

The design of anchorage shown in Fig. 8 is good. The writer agrees strongly with the author's complaint as to the common design of anchorages, which frequently consist of huge tie-bars carried back into enormous masses of concrete out of all proportion to the load which the anchorage is called upon to carry. The writer has known of cases in which it was simple to demonstrate that—entirely apart from the resistances of anchorage of old rails and large washer-plates at the ends of the ties—if the adhesion of the concrete to the tie-bar could be relied upon, it would be possible to break the anchor-bars by tension before reaching an admissible working adhesion between the steel and the concrete. The gate anchorages shown in Figs. 3 and 8 prove that, by rational design, it is possible to construct anchorages of reasonable dimensions which will stand up fully to the requirements of the job. The differentially threaded turn-buckle is ingenious and should enable closer adjustments of the quoin-post to be made relative to the quoin.

The gate-operating machinery shown in Fig. 9 seems well adapted for its purpose, in that it enables a maximum of force to be exerted in the operating arm at the beginning and the end of the stroke, the times when extra effort is most likely to be required. It has the disadvantage, however, that the effort is applied to the gate at a short radius relative to the radius of the center of water pressure on the gate, thereby inducing greater dynamic stresses in the structure of the gate than would otherwise be the case. At the same time, except by the use of the old method of attaching chains at the center of pressure, it is not possible to eliminate entirely the dynamic stress in gates during opening and closing, and the only designs that the writer has seen for substituting an arm for the chain for this purpose had a number of features which were obviously detrimental. It appears necessary to accept the fact, therefore, that compact and convenient gate-operating mechanism can only be obtained at the expense of a certain amount of dynamical stressing of the gate structure.

The writer was interested to note the relatively high degree of elaboration which it has been thought necessary to adopt in designing the structure of the lock-wall where it is intersected by culverts. He questions its value on the generally accepted principle that no structure can be safer than the foundations upon which it stands. The author of this comparatively elaborate graphical and analytical method of designing the wall gives no indication of why he assumes that all of the regular cross-section of the lock-wall acts as an arch. As far as can be followed from the paper, the lock-walls are simply plain mass concrete structures, and it is impossible, without making assumptions of the most arbitrary kind, to know how the pressures on the back and front of the wall are transmitted through the monolithic mass. It is surely going very far to assume that a rectangular wall section with a toe and a circular opening through it, as shown in Fig. 13(*b*), acts as a pure arch composed of a number of independent voussoirs each of special and arbitrary shape. The writer has strong belief in the application of scientific principles to design whenever benefit can be obtained thereby, but he does not think that this particular instance is one suited to extremes of mathematical analysis.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

WATER PRESSURES ON DAMS DURING EARTHQUAKES

Discussion

By Messrs. Theodor von Karman, and Paul Bauman

Theodor von Kármán,[12] Esq. (by letter).[12a]—In his paper Professor Westergaard has presented a beautiful and complete analysis of the pressure distribution on a dam undergoing horizontal oscillations of sinusoidal law. He has investigated both the influence of compression waves in the fluid and the effect of the so-called "apparent mass." It is found that, in the range of the frequencies supposed to occur in oscillations due to earthquake, the effect of compressibility is comparatively small, so that the value of the apparent mass multiplied by the maximum horizontal acceleration gives a first approximation for the excess load due to the earthquake. The following approximate analysis is an attempt at an elementary deduction of the apparent mass, using only the elements of calculus and engineering mechanics.

Consider a unit slice of a dam with a vertical up-stream face, with the height, h, and infinite length, and assume that the dam is at rest when $t = 0$, and that uniform horizontal acceleration, a_x, is acting during the time interval, Δt. To find the exact value of the apparent mass, the exact distribution of the acceleration in the fluid would have to be calculated. Instead of that, it is assumed that a portion of the fluid with a width, b, has the full value of the acceleration, a_x, while the remainder of the fluid is not affected in the process. The width, b, varies with the height (see Fig. 4). With this assumption, Conditions (a), (b), and (c) are readily obtained as follows:

(a) *Condition for "Continuity" of Flow.*—Consider the part of the dam between the bottom at $y' = 0$, and an arbitrary height, y'. Then, the fluid mass displaced by this part of the dam must pass through the section, BC.

Note.—The paper by H. M. Westergaard, M. Am. Soc. C. E., was published in November, 1931, *Proceedings*. This discussion is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

[12] Director, Graduate School of Aeronautics, California Inst. of Technology, Pasadena, Calif.

[12a] Received by the Secretary November 30, 1931.

Assume that the vertical acceleration is variable with the height, and has the mean value, a_y, between $x = 0$ and $x = b$, while for $x > b$, the fluid is

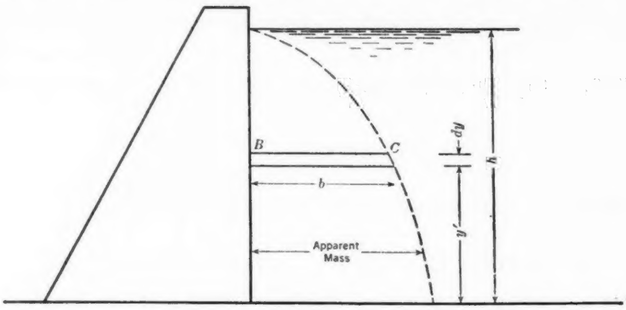

FIG. 4.

supposed to be at rest. The displacement of the dam during the time, Δt, is, obviously, $a_x \dfrac{\Delta t^2}{2}$, and the displacement of the fluid through the section, $BC = b$, is $a_y \dfrac{\Delta t^2}{2}$, so that the condition of continuity requires that $y' a_x \dfrac{\Delta t^2}{2} = b\, a_y \dfrac{\Delta t^2}{2}$, or,

$$y\, a_x = b\, a_y \quad \text{.......................(55)}$$

(*b*) *Equilibrium Between Pressure and Horizontal Mass Forces.*—Let the pressure at an arbitrary height, y', be p_y, and let the density of the fluid be ρ. Then, the pressure is obviously equal to $p\, dy' = \rho\, b\, dy'\, a_x$, or,

$$p = \rho b\, a_x \quad \text{.......................(56)}$$

(*c*) *Equilibrium Between Pressure and Vertical Mass Forces.*—Consider a portion of the fluid body with the variable width, b, between two sections in the height, y', and $y' + dy'$. The resultant of the pressure on the bottom surface of the element is equal to $p\, b$, and that of the pressure acting on the top surface equals $p\, b + \dfrac{d}{dy'}(p\, b)\, dy'$. Now, the difference, $-\dfrac{d}{dy'}(p\, b)\, dy'$, must be equal to the product of the mass of the element, $\rho\, b\, dy'$, and the vertical acceleration, a_y. Hence, $-\dfrac{d}{dy'}(p\, b)\, dy' = \rho\, a_y\, b\, dy'$, or,

$$\frac{d}{dy'}(p\, b) = -\rho\, b\, a_y \quad \text{...................(57)}$$

Substituting $a_y = \dfrac{y'}{b}\, a_x$ from Equation (55) and $p = \rho\, b\, a_x$ from Equation (56), Equation (57) becomes,

$$\frac{d}{dy'}\left[\rho\, b^2\, a_x\right] = -\rho\, y'\, a_x \quad \text{..................(58)}$$

or, since p and a_x have been assumed constant, $\frac{d}{dy'}(b^2) = -y$, and,

$$b^2 = b_o^2 - \frac{y^2}{2} \quad \text{.......................(59)}$$

For $y' = h$, the pressure vanishes, so that $b\ (h) = b_o^2 - \frac{h^2}{2} = 0$, or $b_o^2 = \frac{h^2}{2}$.

Substituting in Equation (59), $b^2 = \frac{1}{2}\left[h^2 - (y')^2\right]$, or,

$$b = 0.707\sqrt{h^2 - (y')^2} = 0.707\sqrt{y\,(2\,h - y)} \quad \text{.........(60)}$$

Hence, with this approximation the shape of the fluid body moving with the dam is shown to be a quadrant of an ellipse. One-half the axis equals $b_o = 0.707\,h$, the corresponding pressure equals $p_o = 0.707\,\rho\,h\,a_x$, or, substituting $a_x = \alpha\,g\,l$ and $\rho\,g = w$,

$$p_o = 0.707\ \alpha\ w\ h \quad \text{.......................(61)}$$

Professor Westergaard's calculation gives,

$$p_o = \frac{8}{\pi^2}\left(1 - \frac{1}{3^2} + \frac{1}{5^2} - \ldots\right)\alpha\ w\ h \quad \text{.............(62)}$$

or,

$$p_o = 0.743\ \alpha\ w\ h \text{.......................(63)}$$

The difference is about 4 or 5 per cent. By the approximate formula (Equation (60)), the total load on the dam is,

$$Q = 0.707\ \frac{\pi}{4}\ \alpha\ w\ h^2 = 0.555\ \alpha\ w\ h^2 \text{..............(64)}$$

While the exact solution gives,

$$Q = \frac{16}{\pi^3}\left(1 + \frac{1}{3^3} + \frac{1}{5^3} + \ldots\right)\alpha\ w\ h^2 = 0.543\ \alpha\ w\ h^2 \text{.....(65)}$$

It is apparent that the approximation is close.

PAUL BAUMAN,[13] M. AM. SOC. C. E. (by letter).[13a]—The author's excellent treatise enables the designer to analyze accurately the increase of water pressure on the face of dams due to earthquakes. Until quite recently this phase of the subject has been a matter of considerable speculation.

In 1928 the writer was making studies on the effect of earthquakes in connection with the statical analysis of a dam in Southern California. While discussing this analysis with Dr. Fritz Zwicky, of the California Institute of Technology, the writer's attention was called to the fact that the inertia of the water might be more important than its flow, which, according to the

[13] Chf. Designer, Quinton, Code Hill-Leeds Barnard, Engrs., Consolidated, Los Angeles, Calif.

[13a] Received by the Secretary December 7, 1931.

analysis, was the only cause of an increase in pressure due to simple harmonic vibrations of the dam. The reasoning was that due to a motion of the dam in an up-stream direction, the water is partly displaced (vertically) and partly compressed. Neglecting for the moment the latter effect and considering the displacement only, the dam acts as a pump, or, better, as a reversed hydraulic ram, forcing the water immediately up stream from it to the

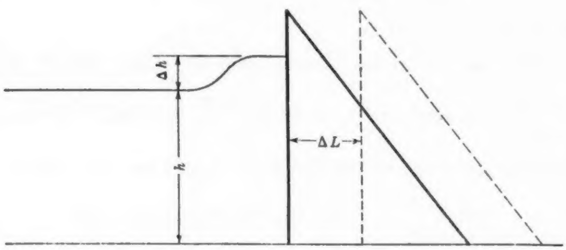

FIG. 5.

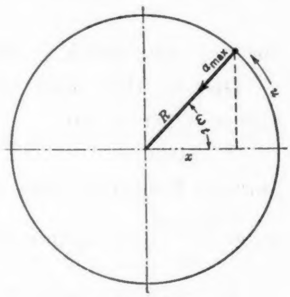

FIG. 6.

surface, as shown in Fig. 5. This displaced water will flow over the top of the dam if the reservoir is full at the beginning of motion, or, if conditions are as shown in Fig. 5, it will flow back into the reservoir.

Principally, the two cases are identical; that is, Δh is the head required for a discharge equal to the maximum volume displaced per unit of time. Thus, if the dam moves at a velocity, v:

$$vh = \tfrac{2}{3}\,\mu\sqrt{2g}\;\Delta\, h^{\frac{3}{2}} \qquad (66)$$

in which (in addition to Professor Westergaard's notation), v = maximum velocity and μ = discharge coefficient. From the law of simple harmonic motion (see Fig. 6):

$$u = \omega\, R = \frac{2\pi R}{T} \qquad (67)$$

$$v_{\max.} = u \qquad (68)$$

$$a_{\max.} = \frac{u^2}{R} = \frac{4\pi^2 R}{T^2} \qquad (69)$$

and,

$$\Delta L = 2\,R \qquad (70)$$

For a period, $T = 1$ sec. and $a_{\max.} = \frac{g}{10}$:

$$v_{\max.} = 2\,\pi\, R \qquad (71)$$

and R follows from $\frac{g}{10} = 4\pi^2 R$, or $R = \frac{g}{40\,\pi^2}$, which checks Equation (7).

The total motion then becomes $2\,R = \Delta\, L = \frac{g}{20\,\pi^2} = 0.159' = 1.9''$.

Introducing $v_{max.}$ in Equation (66):

$$h\,\pi\,\frac{g}{20\,\pi^2} = \tfrac{2}{3}\ \mu\ \sqrt{2g}\ \Delta\, h^{\frac{3}{2}} \quad \text{.................. (72)}$$

and,

$$\Delta\, h = \sqrt[3]{\left(\frac{3\,hg}{40\ \pi\ \mu\ \sqrt{2g}}\right)^2} \quad \text{.................. (73)}$$

for $\mu = 0.80$, and $h = 200$ ft., $\Delta\ h = 8.30$ ft.

Due to this head the pressure increase on a 200-ft. dam would be $\frac{8.30 \times 62.50 \times 200}{2\,000} = 52\ \frac{\text{ton}}{\text{ft.}}$, as against $Q_o = 68.4\ \frac{\text{ton}}{\text{ft.}}$, according to the author's Equation (36); and the moment about the base would be $52 \times 100 = 5\,200\ \frac{\text{ft-ton}}{\text{ft.}}$, as against $M_0 = 5\,486\ \frac{\text{ft-ton}}{\text{ft.}}$, according to Equation (39).

Although the moments about the base check within a few per cent. for this height of dam, it is obvious that the higher the dam the more the physical difficulties grow, involving the movement of water particles all the way from the bottom to the surface within the time of the up-stream motion of the dam. Therefore, it may be stated that although a displacement of all particles immediately in front of the dam necessarily takes place, this displacement is retarded as the depth increases, thereby increasing the volume of inertia, or, in other words, the mass force.

Based on this cognition, Dr. Theodor von Kármán, of the California Institute of Technology, has derived a simple solution of this problem by neglecting the elastic compression of the water and by considering a body of inertia of water, which must be accelerated with the dam and, therefore, offers a resistance equal to its mass times the maximum acceleration of the dam, as also outlined by the author.

Based on Euler's fundamental equation,

$$dp = \frac{w}{g}\ (X dx + Y dy + Z dz) \quad \text{................ (74)}$$

a result has been derived by the writer which leads to an elliptic distribution of pressure and which checks the one due to Dr. von Kármán.

Applied to the problem of pressures on dams during earthquakes Equation (74) reduces to:

$$dp = -\ \frac{w}{g}\ Y dy \quad \text{...................... (75)}$$

As the acceleration, X, in a horizontal direction is constant, $dx = 0$, and the acceleration, Z, parallel to the dam, is zero.

The vertical acceleration, Y, as previously outlined, follows from the condition that the water which is displaced due to a horizontal, up-stream motion of the dam is forced upward. According to Fig. 6 this may be expressed as follows:

$$y\ X = 2\ b\ Y \quad \text{...................... (76)}$$

which is a modification of the condition of continuity as suggested by Dr. von Kármán due to the change of pressure from p at the face of the dam to zero at a distance of 2 b from it. In other words, the body of inertia is actually twice as large as shown in Fig. 6.

The pressure, p, per unit area of dam face follows from the basic law of dynamics, namely:

$$p = m\, X\, b \qquad (77)$$

in which, $m = \frac{w}{g}$ = mass of unit volume of water. Thus, $b = \frac{g p}{w\, X}$.

Introducing Equation (77) and Equation (75) into Equation (74):

$$2\, p\, dp = \left(\frac{w}{g}\right)^2 X^2 \,(-y)\; dy \qquad (78)$$

and integrating:

$$p^2 = \left(\frac{w}{g}\right)^2 X^2 \left(\frac{-\,y^2}{2}\right) + C \qquad (79)$$

For $y = h$, therefore, $p = 0$ and $C = \frac{h^2}{2}$.

There follows,

$$p = \frac{w}{g}\, X \sqrt{\frac{h^2 - y^2}{2}} \qquad (80)$$

which is the equation of an ellipse. Numerical evaluations of Equation (80) check those due to the author's Equation (52) within a few per cent.

Finally, due to Equation (77):

$$b = \sqrt{\frac{h^2 - y^2}{2}} \qquad (81)$$

The author's efforts throughout the paper to make the same theory comprehensive to engineers who cannot be expected to be "high-powered" mathematicians is quite apparent. It is very thought-provoking indeed, and deserves both commendation and imitation.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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DISCUSSIONS

LOW-COST BITUMINOUS ROADS

Discussion

BY MESSRS. H. J. SPELMAN, J. T. L. MCNEW, ROGER M. LEE, E. Q. SULLIVAN, AND HUGH W. SKIDMORE

H. J. SPELMAN,[5] M. AM. SOC. C. E. (by letter).[5a]—This paper is an interesting and valuable review of present practice in the Western States in the bituminizing of crushed rock and other traffic-bound surfacings. Mr. Frickstad discusses three classes of such treatments, namely, surface treatments, road mixes, and plant mixes. The total of 12 000 miles completed in those States indicates the importance of the subject.

There would appear to be no reason why the road-mix and plant-mix types should not approach in wearing qualities the asphaltic concrete, Topeka, or sheet asphalt surfaces, the characteristics of which they approach more or less closely.

There are two points in particular, which it is believed should be specially stressed. The first is the importance of sufficient manipulation or spreading, both in plant mixes and road mixes, to insure smooth riding qualities. If plant mixes are to be used more and more, as indicated in the paper, this is particularly important if this type is to give general public satisfaction. The second point is that failures are due principally to lack of sufficient base. It is an old, old story that most failures of pavement or surfacing are due to that cause, and it is well to emphasize that these new types will not take the place of bases of adequate thicknesses.

No mention is made in the paper of the so-called retreads which have been developed quite extensively in some of the States east of the Mississippi River during the time that these oil mixes have been developed in the Western States. As the types covered by Mr. Frickstad compare in characteristics with the older sheet and asphaltic concrete, so do these retreads

NOTE.—The paper by Walter N. Frickstad, M. Am. Soc. C. E., was presented at the meeting of the Highway Division, Sacramento, Calif., April 24, 1930, and published in November, 1931, *Proceedings*. This discussion is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

[5] Washington, D. C.

[5a] Received by the Secretary December 2, 1931.

compare in characteristics with the so-called, mixed bituminous macadams of 20 to 25 years ago, or with the "cold-patch mixtures" used so long for patching almost all kinds of pavements.

These retreads require no material less than $\frac{1}{4}$ in. in size, and the usual size is from $\frac{1}{2}$ in. to $1\frac{1}{2}$ in. Like the oil-mixed surfaces discussed by Mr. Frickstad they are constructed either by the road-mix or plant-mix method. Their increased cost over the Western oil mix is probably due only to the cost of the coarse aggregate which is hauled in and placed on the road just in advance of operations. Cut-back asphalts and tars have both been used successfully in their construction and more recently the development of a slower setting emulsion has enabled that material to be used with excellent results.

This type of construction has proved of great value in the East in the smoothing of old rough macadams, as well as in providing low-cost, dustless, wearing surfaces for the traffic-bound types. It is probable that its development was largely brought about because of the larger number of commercial plants ready to furnish the needed coarse aggregate of crushed stone, slag, or gravel. As in the Western types the riding qualities of this road-mix type are equal to those of any pavement. Failures have been generally attributable to the base, and plant mixes have not produced as smooth surfaces as road mixes. The smooth riding qualities of this type of construction are due primarily to the use of heavy road machines and heavy drags, which has been made possible by the development of tractors of the heavy, crawler type. If plant mixes are to compare favorably in riding qualities with road mixes, the use of similar methods of manipulation is essential.

The dividing line between surface treatments and retreads is not particularly well defined; nor is the term, "retread," a particularly satisfactory one for this type of construction. The term, "road mix," is a better one. The class of road mix used in the Western States might be called "fine aggregate road mix," and that used in the Eastern States, "coarse aggregate road mix."

Certainly, the dragging of all surface treatments is highly desirable, in order to obtain smooth riding surfaces. A suitable method of differentiation between retreads or road mixes and surface treatments might be to name as "retreads" or road mixes those jobs that involve the use of road machines, harrows, or other mixing equipment; and to call those surface treatments, that involve only dragging and rolling.

Mr. Frickstad's contribution is particularly important and timely because the increasing speeds of all classes of vehicles, as well as the increased number in use, have caused the road users everywhere to demand not merely a usable, all-year road, but a dustless, usable, all-year road.

J. T. L. McNew,[6] M. Am. Soc. C. E. (by letter).[6a]—Experiences of Western engineers with so-called "low-cost" bituminous roads have been summarized briefly in this paper. The term, "low cost," may be open to the

[6] Prof. of Highway Eng., Agri. and Mech. Coll. of Texas, College Station, Tex.

[6a] Received by the Secretary November 27, 1931.

objection that poor alignment, grade, and workmanship are to be permitted to accomplish "low cost." Invariably, the term is taken by the client to mean small outlay of money, and, in time, it may defeat the real purpose which engineers have had in mind as these surfaces for moderate traffic have been developed. Engineers are agreed that alignment, grade, and structures are probably the most permanent features of a highway; that safety to traffic demands that curves be flattened, widened, and superelevated; and that sight distance must be provided. They also recognize the importance of establishing grades to meet the demands for convenience of the motoring public. These demands must be met almost without regard to surface, and when so met they form a part of the permanent investment. "Low cost," therefore, is not to be applied to these appurtenances, except perhaps in the case of drainage structures themselves.

The term as applied to the surfaces then remains as the subject, and even yet it is somewhat misleading to the client. Much data are available to show both success and failure of so-called "low-cost" surfaces, and usually the promoter who reaches the client first uses the data most convenient to carry the question in favor of his particular product or type of surface.

Engineers must soon make public their intentions of recommending, designing, and constructing road surfaces to withstand present traffic requirements, plus the requirements of any increase in traffic that may reasonably be expected within a few years. Naturally, then, first cost, maintenance cost, amortization costs, transportation costs, and salvage value all must be considered in type selection to meet the needs of traffic in a locality. When, and only when, engineers accept and begin practicing these fundamental principles will the public begin to get the maximum possible mileage of serviceable roads, and only then will it recognize the fallacy of the idea so often expressed that "this or that surface is permanent." A highway is a structure that must be improved progressively as traffic and maintenance costs demand, and if roads continue to be needed for transportation purposes, they must be rebuilt time and time again in the future.

Bituminous surfaces have been brought to the attention of engineers rather forcibly in recent years and although several different methods are in use they are, in the final result, very similar. Much research has been reported, giving valuable information and guidance, and much more remains to be done.

The mention of the question of asphalt content invariably induces discussion. There are those who regulate the quantity by empirical rule; those who proportion by eye; those who insist that the stain test determines the quantity; and those who proportion the bitumen according to the voids in the aggregate. Naturally and logically the characteristics of the oil have a bearing upon the question as to how much bitumen the surface mixture must contain. Time (or perhaps it might be expressed as "age" of sample) also is a factor, because the light oils or topped crudes must certainly weather or volatilize more than the heavier grade oils. If, by bitumen content, is meant that content at the time the mixture is opened to traffic, and if any rule of proportioning is adopted for two mixtures containing different oils, then

these two mixtures are alike in content at the beginning, after which they show variations in bitumen content dependent on the traffic using the surface and the climatic conditions as well. In general, the higher the percentage of asphalt of 80–100 penetration that an oil contains, the more stable it will be under atmospheric conditions and the more nearly true the given proportioning rule will be found.

The difference of opinion between the stain-test advocates and the devotees of the voids theory are not really as great as may appear, for each theory has its correct place in modern practice. Stability tests made on paving mixtures invariably indicate that, when bitumen in excess of that necessary to fill the voids in the compacted mineral aggregate is used, the stability as measured by resistance to shear is decreased. With these results the voids-theory enthusiast draws the conclusion that his theory is correct. On the other hand, the proponent of the stain test (which is no doubt a modification of the pat test as described by Mr. Clifford Richardson[7]), insists that the particles of aggregate should be coated with bituminous binder. Unfortunately, however, if the aggregate is proportioned for low voids in the compacted aggregate this coating of bituminous binder, as determined by the stain test, will prove to be an excess over that required to fill the voids, with the result that the stability test will indicate a weak mixture. If the aggregate contains more voids, then the stain test does not give an excess over that required for voids; hence, stability as affected by bituminous content will not be impaired. Of course, the voids theory must be limited for use with mixtures of aggregate that have sufficient voids to accommodate asphalt in an amount that will impart another property to the surface, which is perhaps more important than any other, namely, that which may be called "flexibility," or, perhaps, "resistance" to suddenly applied forces.

The writer has been conducting investigations in an attempt to find the critical bituminous content of certain paving mixtures below which excessive brittleness or lack of flexibility may be expected. The data accumulated from this work indicate that maximum resistance to displacement caused by suddenly applied loads invariably requires more bitumen content than is needed for maximum stability under static or gradually increasing load, as used in the various stability tests.

As the author has indicated, proportioning by empirical rule or formula has value only as a means of estimating bituminous content. In presenting the formula for the percentage of oil (Equation (1)),[8] C. L. McKesson, M. Am. Soc. C. E., stated: "It is not applicable to porous or absorbent materials, such as cinders or lava, but otherwise it appears to give results which are consistent with service obtained from various mixes." The writer wishes to emphasize also that, perhaps, Equation (1) will not apply to oils different in characteristics. Some oils contain fractions of highly volatile compounds, whereas others contain practically no very volatile fractions, and the different oils are absorbed in different quantities by a given aggregate. It must always

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[7] "The Modern Asphalt Pavement", by Clifford Richardson.

[8] "Light Asphaltic Oil Road Surfaces", *Public Roads*, September, 1927.

be kept in mind that the term, "oiled road," may be applied alike to surfaces that may contain crude oil, topped crudes, fuel oil, residual oils, emulsions of any of the heavy residuals, or cut-backs, and that the determination of the proper bitumen content should be predicated upon one of the proper quantity of the particular oil to be used.

Mention has been made previously of the fact that, dependent upon the oil used, the bitumen content will decrease with the passage of time and will be influenced by both the climatic condition and amount of traffic the surface carries. Bituminous surfaces that carry light traffic usually show higher losses of the volatile oils than those under heavy traffic.

Incidentally, the intensity of the maintenance may affect the choice of bitumen content. If a surface is to receive a mixed-in-place treatment of a cut-back, or light topped crude, and provisions are made to maintain the surface continuously, one would be justified in using a slight excess of oil with the knowledge that more maintenance would be required to keep the surface smooth until the oil has reached a more stable condition upon exposure to traffic and weather. The engineer will probably recall instances where "shoves" have occurred in bituminous pavements soon after the surface was opened to traffic and that later these places are the most stable spots in the surface and seem to hold up under the more severe impacts. Not all engineers recognize the fact that a bituminous surface does move about under traffic and that a period of readjustment does take place. During this period the base itself may settle or heave, and careful maintenance in early life is essential to insure a smooth surface. It is also during this period of adjustment that "self-healing" is a desirable characteristic of the mixture. The ability of a surface to rise or fall with the subgrade, and to heal over the crack formed, is dependent upon the content of bitumen in the mixture, that has real binding properties. Experience has taught engineers to evaluate an oil on this score by its asphalt content of 80–120 penetration.

At times, one hears the statement made that the thinner the coating of the gluing agent the stronger will be the bond between two particles stuck together; and then the statement will be used as an argument for extremely lean mixtures. It is admitted that as bitumen content decreases to a given point, stability against shear as measured by static load increases, but the same law does not hold with the same quantities of bitumen if the stability is measured under impact loads. Bituminous surfaces do not have beam strength and no sacrifice of bitumen will impart sufficient strength to allow it to carry load as a beam.

If bituminous surfaces cannot be made to have slab strength sufficient to allow the surface to span soft spots when subjected to load, such facts should be admitted and a proper foundation prepared that will distribute the load over an area of such size that the subgrade will not be over-loaded. A sufficient quantity of oil should always be included, in order that the surface may follow the settlements of the subgrade or foundation course without cracking and subsequent disintegration.

The author calls attention to the fact that no standardized grading for aggregate for low-cost surfaces has been adopted. In the light of present

knowledge of bituminous pavement practice perhaps no standard will be adopted. Surfaces falling naturally into the class of "low cost," for the most part, have been made of local materials on account of limited funds. Undoubtedly, the success of the methods of constructing these surfaces with various gradings of aggregate is evidence sufficient to disclose the fact that there are many satisfactory gradings. It is only within recent years that asphalt technicians in the field of hot-mix pavements have disclosed the fallacy in the assumption that any one grading is necessarily the best and that all others must be discarded. To standardize the practice and accept only one grading would defeat the element of "low cost" remaining. Concerning the theory of low-void aggregates there follows the fact that any number of combinations of mineral aggregate can be made with reasonably low total voids and minimum size of individual voids.

Mr. Frickstad states that the part of the aggregate passing the 200-mesh sieve is an important element, and the writer is fully agreed that such material of proper kind is beneficial. At the same time, attention should be called to the fact that this 200-mesh material must be controlled very carefully as to quality, such choice to be dependent upon the climatic conditions to which the surface is to be exposed.

Unfortunately, clay, the worst soil with which highway engineers must deal, passes the 200-mesh sieve. Certain classes of clay have the greatest attraction for water, the largest shrinkage factors, the poorest bearing values, and the highest absorptive properties of soils. Incidentally, some clays have greater affinity for water than for asphalt or tars. An aggregate containing appreciable quantities of such material is of doubtful value for districts where seasons of heavy rain exist. In some sections of the West and Southwest good results have been obtained with aggregates containing clay up to 25 per cent. The success of these surfaces has been due no doubt in part to good drainage for occasional rains and the absence of prolonged wet weather. Excess clay in oiled surfaces, in localities where rainy reasons are prevalent, usually makes its presence known as an extremely slippery surface and one in which the asphalt apparently emulsifies during the rain and immediately afterward.

The quantity of 200-mesh material, as well as the quantity of aggregate passing the 10-mesh, must be used somewhat as a guide as to the choice of oil. Aggregates containing "fines" similar in amount to the Topeka mixtures are much more difficult to mix uniformly with the heavier oils of 80 to 90% asphalt. Aggregates containing such quantities of oil are readily miscible with light oils, emulsions, and cut-backs, and are likely to require more care during the curing period. The writer has used the Topeka class of mixtures with some success on maintenance work by first introducing a distillate to moisten the aggregate in the mixer and then adding the heavier oil. As indicated previously, however, the mix does not reach good stability values until time enough has elapsed to allow a considerable quantity of the volatile fraction to evaporate. This method, therefore, requires careful maintenance until the surface has "set."

The writer feels confident that the experience of the Western and Southwestern States with these so-called "low-cost" surfaces has brought a means of extending the mileage of serviceable roads and has done much to focus the attention of taxpayers and engineers on the undisputable fact that highway-surface betterment is a problem to be solved by progressive improvement wherein each successive stage of development will realize the maximum salvage value of the last surface. It must be admitted, however, that there are yet many problems to be solved with these low-cost bituminous surfaces and that the acceptance of any set rule of mix design, bituminous content, or other construction method, without the proper care to see that such rules are acceptable for the case at hand, is open to numerous pitfalls to the disadvantage of the constructive work that Mr. Frickstad and his Western associates have so successfully carried to the present stage of development.

ROGER M. LEE,[9] ESQ. (by letter).[9a]—It is interesting to note that surface treatment of the type described by the author has been applied to more than 12 000 miles of roads in eleven Far Western States. Experience thus gained will be of very great value.

In Brant County, Ontario, Canada, on secondary roads tar and emulsified asphalts have been used, the latter being either plant-mixed or applied in the usual manner. Preferably, a little more money is spent to obtain a retread or, better still, a gravel asphalt is put through a standard asphalt plant and sealed with hot, mixed chips applied along with the remainder of the material so as to prevent the possibility of a slippery surface. With a reasonable base of gravel, 3 in of this surface would apparently provide fairly permanent results.

E. Q. SULLIVAN,[10] ASSOC. M. AM. SOC. C. E. (by letter)[10a]—One of the first road surfaces constructed by the California State Highway Department, by the oil-mixing treatment, was 44.3 miles between Victorville and Daggett, in San Bernardino County. This road was originally surfaced in 1927 with a 4-in. thickness of crushed gravel, with an average grading approximately as follows:

Sieve	Percentage passing	Sieve	Percentage passing
Fuel oil	3.7	20	45.9
200	10.6	10	56.4
100	19.1	3	72.4
80	22.8	$\frac{1}{2}$	82.6
50	28.9	$\frac{3}{4}$	93.3
40	33.5	1	100.0
30	37.8		

[9] County Engr. and Road Supt., Brant County, Brantford, Ont., Canada.

[9a] Received by the Secretary November 25, 1931.

[10] Dist. Engr., State Div. of Highways, Dist. VIII, San Bernardino, Calif.

[10a] Received by the Secretary November 30, 1931.

The surface was not oiled originally. After this road had been under traffic for one year, there was considerable loss of surfacing material, the exact amount being unknown as the subgrade was sandy and gravelly and some of it had worked into the surfacing. The road surface had not started to fail when it was oiled, but was still in fairly good condition.

Oiling was undertaken by the oil-mixing treatment described by Mr. Frickstad. The cost per mile was, as follows:

Oil furnished and spread	$577.76
Mixing	498.84
Oil for seal coat	94.51
Labor, etc., for seal coat	60.95
Total cost per mile for oil-treating the completed road	$1 232.05

This road has been under an increasing volume of traffic, which has averaged, according to 1931 traffic counts, about 70 trucks per day, plus 850 automobiles, or a total traffic count of 920 vehicles for the 16-hour period between 6:00 A. M. and 10:00 P. M. The average maintenance cost has been as shown in Table 1.

The condition of this road surface has been kept continuously good, including the edges of the surfacing. Since the original gravel surface was only 18 ft. wide, the road is rather narrow for the increasing width of automobile stages and trucks using it. The increased cost of maintenance is due to increased expenditure on the edges of the road surface. Other maintenance has consisted of re-mixing a few short stretches of road surface that showed signs of being too rich in consistency, with resulting corrugations. Probably not more than 1% of the total road surface has been re-mixed as a maintenance measure.

TABLE 1.—SUMMARIZED COST PER MILE FOR MAINTAINING VICTORVILLE-DAGGETT ROAD

Year.	Road surface cost per mile.	Total cost per mile.
1927–28	$51.54*	$323.63
1928–29	92.46	174.05
1929–30	171.37	293.20

* Estimated; not accurately separated from total cost in this year.

A few broken places developed in this road surface during the first three years, perhaps an average of one such place per mile. These broken places were due, in general, to rodents that excavated burrows, thus causing failures 1 ft. or 2 ft. in diameter. The cost of such repairs is very small. The major maintenance work has been to hold the edges of the mixed surface from breaking.

Experience with this route indicates that with the present traffic it can be maintained indefinitely at about the cost indicated. Evidently, the cost

of maintenance could be reduced materially if the road surface were widened to eliminate the tendency of wide vehicles to run off and on the edges, breaking them down.

Should the traffic increase on this road and more heavy trucking become established, it might be necessary to re-work the road at some future time, but it has been found by experiment that this road surface can be remixed to a greater thickness at small cost.

The road is constructed in a semi-arid country, on a well-drained sub-base of natural sand and gravel. In view of its success and the apparent permanent nature of the road surface, it was decided to extend the work across similar arid or semi-arid country where good, well-drained sub-base is available.

Alternate bids for field-mix *versus* plant-mix were called for in the first contracts. It was found that contractors bid approximately the same for either method of work, and subsequent calls for bids provided only for plant-mixing of the road surface.

The project provided for the construction of the subgrade by shaping, watering with a sprinkling truck, dragging, and then encouraging traffic to compact it for a period of two weeks. All contract work provided for a similar method of constructing the subgrade. This method is well suited to arid conditions; natural materials in arid regions usually cement and become very hard under this treatment. Exceptionally smooth riding qualities are attained in the profile by this subgrade construction when long drags are used.

The standard of work has been increased in later projects in view of the expectation of heavier traffic. The road constructed under the first contract has a thickness of from 2 to 3 in., but all contracts let have provided for a loose thickness of 5 in., which results in a compacted thickness of about 4 in.

The cost of the Victorville to Daggett project does not include that of crushing and spreading the original crushed gravel surface or the cost of the subgrade, but the cost of the subsequent projects in Table 2 includes that of crushing the gravel, plant-mixing the material, furnishing the oil, constructing the subgrade as outlined herein, spreading the material on the subgrade, windrowing the material, and spreading and laying it down for traffic to compact.

Table 2 is an analysis of the cost of seven typical projects covering 133 miles of road. These were all new roads, the surfacing being placed on new alignment on a newly completed grade. The cost of grading and structures is not included in this analysis. The cost of water used in sprinkling the subgrade has been separated from the remainder, because conditions vary so much in this respect in arid regions. The water for some of these projects was hauled in tank cars on the railroad for a distance of 60 miles. On other projects the contractor was able to develop water by drilling wells.

It has been found necessary in District VIII of the California State Highway Department, to surface-treat all projects with fuel oil and screenings. In general, it is most desirable to make the mix so dry that there

is a tendency to ravel without this surface treatment. The desirable feature in requiring a dry mix lies in increased stability, resulting in elimination of any tendency of the road surface to creep or corrugate; the possibility of bleeding is also eliminated.

The treatment has been applied immediately after completion and compaction of the road surface. The cost of this treatment has averaged $155.46 per mile on day-labor work.

TABLE 2.—ANALYSIS OF COST OF SURFACING ON SEVEN TYPICAL CONTRACT JOBS COVERING 133 MILES OF ROAD

Location.	Miles.	Cost of Water for Sub-Grade.		Cost of Surfacing in Place, Including Oil and Seal Coat.		
		Total.	Per mile.	Total.	Per mile.	Per ton.
1½ miles northeast of Yermo to 1½ miles southwest of Dunn. .	20.78	$6 885.10	$331.33	$83 791.75	$3 998.04	$1.63
4 miles west of Hector to 2 miles west of Argos................	14.00	4 232.00	302.29	63 810.50	4 557.89	1.72
Daggett to 4 miles west of Hector.	21.22	4 361.70	205.55	101 728.14	4 793.97	1.78
Barstow to 1 mile east of Yermo.	13.05	7 040.00	539.47	60 659.50	4 648.24	1.78
1½ miles west of Siberia to 6½ miles east of Amboy.........	22.38	17 500.00	782.30	130 437.60	5 830.92	2.19
9½ miles west of Hopkins Well to Black Butte................	22.10	8 542.90	386.56	133 375.78	6 035.10	2.36
2 miles west of Argos to 1½ miles west of Siberia..............	19.47	16 000.00	821.78	123 045.00	6 319.72	2.38
Averages for 133 miles of construction..................	18.998	$9 223.10	$481.32	$99 549.75	$5 169.12	$1.98

These roads, completed by plant-mixed methods, have given an opportunity to analyze reasons for best results. Experience in California to date indicates that there are three major points governing the attainment of best results, namely, workmanship, quality of oil, and quality of aggregate.

Workmanship.—It has been found that even when the quality of materials is not of the best, reasonably satisfactory results can be obtained if additional skill and effort are put into the workmanship. By this is meant, road-mixing and remixing, with careful adjustments in the quantity of oil.

Quality of Oil.—Plant-mixing has clearly brought to light the fact that, of several oils passing California State specifications, there is a marked difference in results.

Quality of Aggregate.—It has been found that the quality of aggregate influences results, not so much because of qualities of hardness of the particles as qualities of natural binding value in the aggregate. Clay in the aggregate, however, is found to be detrimental. In general, an aggregate having the best mechanical lock, will make the best aggregate for oil-mixing treatment.

It is believed that, if desired, the plant-mixed roads can also be remixed by field methods at any time, with any desired thickness of material added to increase the strength for possible increased stress of heavier traffic.

Conclusion.—The oil-mixing treatment for graveled roads is particularly suited to arid and semi-arid regions, where such roads are subjected to reasonable loads and to moderate traffic. Maintenance costs are low under these

conditions. The traffic that these pavements can withstand is as yet undetermined since there is no sign of general failure. The strength can be increased to meet increased future demands.

There were two major developments in construction methods during 1931. The first was the discovery that heating the aggregate on plant-mixed jobs results in a saving in cost. The majority of contractors installed and used heating equipment at the plant, although this was not required under the specifications.

Two major savings are affected by heating the aggregate: (1) The power required to turn the pug-mill is greatly reduced; and (2) plant production is increased because the more rapid coating of the particles of the aggregate with oil reduces the time of mixing.

The second major development of 1931 was the marked improvement in riding qualities of plant-mixed surfaces. Previously, most engineers believed that it was impossible to secure, on plant-mixed jobs, the ideal riding qualities of the road-mixed jobs. It is now found that the same ideal riding qualities can be secured by requiring an appropriate amount of blading of the plant-mixed material. Contractors' bids indicate that there is no appreciable increase in cost in specifying a definite amount of blading with relation to plant output.

HUGH W. SKIDMORE,[11] ASSOC. M. AM. SOC. C. E. (by letter).[11a]—Prime desiderata of low-cost road development are: Mileage, serviceability, and cost. The aim is to secure the most miles of the most serviceable road at the least possible cost. A logical development of this idea naturally lends itself to "stage" construction in accordance with the demands of traffic, and entails the utilization of local aggregate materials and, most generally, some one of a variety of bituminous binders.

Low-cost road construction has developed rapidly during the past few years due to recognition of the economic fallacy of expensive pavements on secondary roads and, as yet, rather sparsely traveled sections of trunk highways. A very great mileage of highway pavement, costing from \$25 000 to \$50 000 per mile, was unwisely built in many sections of the United States before low-cost road construction reached its present widespread application and before lack of funds forced economies in highway expenditures. One of the benefits of the present financial depression to the highway construction industry is the impetus it has imparted to low-cost road development mainly through the recognized necessity of securing greater road mileage per dollar spent.

The general scope of low-cost road construction is quite broad, both geographically and economically. Every section of the continent is deeply concerned in its application and a considerable number of localities have progressed to a quite well standardized procedure. With respect to cost, a wide variety of types and thicknesses, ranging from \$1 000 to \$15 000, or more, per mile, may quite properly be included, as compared with expendi-

[11] Pres., Chicago Testing Laboratory, Inc.; (Craig, Skidmore & O'Brien, Inc.). Chicago, Ill.

[11a] Received by the Secretary December 11, 1931.

tures of from $25 000 to more than $50 000 per mile for so-called "permanent" types. In many instances the latter types are not living out the bond issues under which they were constructed, or they have reached a condition in which the maintenance cost is prohibitive. Resurfacing of these originally expensive pavements constitutes a separate field of application of low-cost surfaces which is likewise developing rapidly.

The exact nature of the composition used in low-cost road construction is entirely a matter of locality and the materials available therein, considered in the light of the traffic to be served. Thus, in certain sections of Nebraska, blow sands are the only locally available aggregates, while in other sections of the State a very finely graded gravel (almost a coarse sand, known as Platte River gravel), is the only aggregate available. Some States are blessed with an abundance of both gravel and durable stone, while in some instances very large districts are almost devoid of any kind of suitable aggregate. The Gulf States have available large quantities of oyster shells. Hence, it is clear that the development of pavement or treatment type is peculiarly a strictly local problem, and several varieties may be required within a given State.

A wide variation in aggregates entails a wide range in bituminous binders, from road oils, which in some cases are little more than topped crudes, to highly refined asphalts. Tars also include a wide range of consistencies from very light, volatile, priming tars to heavy, viscous, hot-application binders. Some binders, both tar and asphalt, are cut-back or diluted with a more or less volatile solvent (depending upon the nature and gradation of the aggregate) to enable the use of heavy, viscous binders at air temperatures, or slightly above, allowing the diluent to evaporate rapidly or slowly as the case may be. This evaporation begins during construction as soon as the bitumen is applied to the aggregate and continues for short or long intervals after construction, depending upon the volatility of the diluent used. Likewise, both tar and asphalt are emulsified with water which forms a non-adhesive, mobile liquid, enabling them to be used cold. Some time after application, the emulsion breaks, freeing the water and causing the composition to "set;" that is, the bitumen resumes its normal state of adhesiveness. By this means, quite viscous bitumens may also be utilized without using heat. Emulsions possess the added feature of being applicable to damp aggregate and, in fact, they often function best upon moist surfaces.

While a great variety of aggregates exists, there are also many types of binder available so that practically any aggregate condition may be met. The selection of the most suitable bitumen is not always immediately apparent except in a general way. Traffic, climate, and aggregates are the principal determining factors, but frequently considerable experimenting is required and some modification of existing binders is indicated. Naturally, intimate knowledge of bituminous materials is highly specialized and the application of such knowledge to the problem is essential to economic success.

The simple surface treatment of roads with some form of oil or tar will perhaps always be more or less common, solely as relief from the expensive dust nuisance. Most highway officials will freely state that such treatment is easily warranted for this reason alone, to say nothing of the preservation

of road surface and metal afforded thereby. It is not at all uncommon to find gravel and stone road surfaces losing the equivalent of an inch of metal per year from wear, when untreated.

Surface applications are not permanent, however, and require more or less frequent renewals. This may become burdensome and then a more substantial pavement surface is indicated. Road mix or plant mix, utilizing some one of the several cold application binders, quite generally follows surface treatment when it becomes inadequate or too expensive. Conditions may justify even more substantial construction, such as bituminous concrete or sheet asphalt. It is certain, however, that the next decade will witness an increasingly vast mileage of simple, inexpensive, cold bituminous mixtures. Recognition of this fact has resulted in numerous patented types and processes, some of which have merit. Realization of the progress to come in this field has also encouraged manufacturers to apply their engineering skill to the development of equipment which will not only enable more miles to be built for less money, but will insure better pavement by eliminating the human element with respect to uniformity of mixture composition and smoothness of road surface.

Designers, builders, and users of pavements all desire smoothness of surface initially, and as long thereafter as possible. The engineer, aside from professional pride in his creation, realizes that smooth surfaces which retain their smoothness eliminate a costly source of maintenance; the builder, from a plain business point of view, knows that a superior surface is an important aid in securing additional work, and if he is in any degree responsible for maintenance, as is frequently the case for at least the early life of the pavement, he, too, appreciates the effect of initial smoothness upon subsequent expense; the user, obviously, is most intimately and frequently apprised of poor riding quality and he is seriously concerned as a taxpayer in both initial and ultimate cost. Of these three individuals, the engineer, in the main, must assume the burden of improvement and he is, therefore, especially interested in new methods, new materials, and new machinery that may offer possibilities of superior workmanship, greater durability, and reduced cost.

Any bituminous paving composition, regardless of hardness, will flow under pressure in proportion to the load applied and the length of time it is applied. Thus, resistance to distortion is mainly resident in ability to withstand shearing forces. In those compositions utilizing very soft binders, such as oils, very soft asphalts, and tars, stability is principally dependent upon close interlocking of aggregates since the binders are relatively fluid at normal temperatures. The more closely graded and better filled the mineral aggregate, the more effective, from the standpoint of stability, are very soft bitumens. This is another way of stating that as cements, they are more effective in thin films on closely packed particles than in thick films on aggregate masses containing large void spaces and relatively few contacting surfaces.

One of the major types of composition used in low-cost road construction is the rather open, coarse-graded type with mineral (either uncrushed gravel, crushed stone, slag, chatts, or gravel) coated with tar or asphalt, cut-back, or

emulsified asphalt or tar, and sometimes a heavy asphaltic oil. There are numerous modifications of this general type, including variations in aggregate sizes from all coarse, large-sized particles; graded coarse particles exclusive of fines; graded stone or gravel plus moderate quantities of fines (sand or stone screenings), to rather well filled graduations, resembling graded bituminous concrete in texture. Binders requiring volatilization necessitate the use of fairly open mixtures with comparatively large void spaces; they are sealed with a surface application of liquid bitumen and chips, sand, or pea-gravel top dressing following an interval of time for evaporation of the diluent. When such binders are used in too dense a composition, volatilization is retarded or prevented to the extent that a crust forms at the surface, preventing complete volatilization in the lower portion of the mixture. Thus, cut-back binders and volatile oils are more or less restricted to open mixtures for the best results. Emulsions, however, can be successfully used in filled mixtures which develop ample stability to permit immediate compression and use by traffic. Suitable oils may be similarly used, but are limited as to the degree of stability developed in the mixture.

Mr. Frickstad confines his discussion to oil-mix and oil-surface treatment in the Far West, recognizing that there are many other similar and entirely dissimilar developments in other sections. Perhaps no section of the country has witnessed such extensive and varied application of low-cost road types as the Central States where practically the whole gamut of road mixes, plant mixes, and surface applications has been run, using such binders as light and heavy oils of asphaltic, semi-asphaltic, and non-asphaltic nature; tars, asphalts, cut-backs, and emulsions; and such aggregates as soils, sands, gravel, stone screenings, slag, chatts, and stone. Furthermore, a great number of proprietary formulas have been developed covering cold-mix, cold-lay; hot-mix, cold-lay; and cold-mix, hot-lay ideas, using all the various aggregates and bitumens. Some of these compositions depend upon the blending of hard, brittle asphalts with soft fluxes (oils) under the action of traffic. The real status of the matter is that this field of construction is just nicely started and outstanding developments, both with respect to cost and durability, may be counted upon during the next few years.

All engineers, and contractors as well, recognize that the greatest single influence in the direction of lower costs is the substantial increase in the production capacity of a given crew of workmen and quota of equipment. Obviously, then, real progress in this direction calls for the application of the inventive ability and mechanical ingenuity of equipment manufacturers. These men have not been slow to realize this fact and have wisely reaped the benefits of co-operation with engineers and contractors in developing machines of practical value. Every engineer realizes that improvements in methods and equipment are vital to real progress; therefore, it is surely proper that sincere recognition be given the designer and builder of machines which will give a better product at a lower cost.

During 1931 machines of the type shown in Fig. 3 have been developed. They have a wide application in the building of better compositions at substantially lower costs than have yet been possible. To the engineer or con-

tractor who observes this equipment in operation, it is immediately apparent that plant mix of the very best quality can be manufactured, spread, and finished mechanically with much less effort and equipment, in greater quantities, and with positive assurance of far superior workmanship, than has been possible by road mix or plant mix to the present time.

The equipment comprises the application of well-known principles which eliminate the personal equation and insure uniformity of composition, perfect coating of aggregates, and a finished pavement of smooth surface and uniform denseness in all portions of the roadway. These very desirable characteristics may be obtained by the use of the full range of aggregates and binders, and upon any subgrade or base which is suitable to function in such capacity. The adaptability and flexibility of the equipment are two of its outstanding features.

The first unit consists of a large capacity, pick-up loader, with spiral feed, discharging into an overhead bin which, in turn, discharges upon a positive-feed, corrugated apron conveyor that has an adjustable, calibrated discharge which gauges, accurately, the continuous flow of aggregate. From the apron feeder, the aggregate drops through a baffled chamber where the bitumen is applied either hot or cold by means of two groups of high-pressure nozzles. One set of nozzles spray the falling mineral as it leaves the apron, the other group as it leaves the baffles and just before it reaches the large-capacity, twin pug-mill mixer. These sprays are effective in coating at least two-thirds of the total surface.

The flow of bitumen is accurately controlled according to requirements by valves and a recording flow-meter. When the material reaches the mixer, it is already fairly well coated and by the time it has traveled the 8-ft. length of the twin pug-mill, it is thoroughly and uniformly coated. The machine has a capacity of $2\frac{1}{2}$ cu. yd. of aggregate per min., operating at normal speed. The flow of mineral and bitumen is set according to the requirements of the mixture, and remains constant in this ratio until changed to accommodate a new condition. Thus, through continuity of flow of materials and synchronization of elements, exceptional uniformity of product is maintained.

With the loader mixer operating on the subgrade picking up the aggregate from windrows, the full benefit of sun and wind in drying the mineral is secured, thus eliminating costly mechanical drying which, if provided in sufficient capacity to supply the mixer adequately, would be prohibitive in many cases. Operating as a central plant at the source of aggregate, mechanical drying may be readily provided, but always with a substantial increase in cost of mixture.

The second unit is the spreader finisher, which also operates under its own power. Both machines are mounted on crawlers and are adequately powered to provide ample traction under the most adverse conditions. The finisher may operate directly behind the mixer with the mixture discharging directly into the spreader. The latter consists of two horizontal screw conveyors delivering mixture the full width of the road and in any finished depth up to 6 in. The screws operate independently so that the quantity may easily be controlled according to the condition of the subgrade.

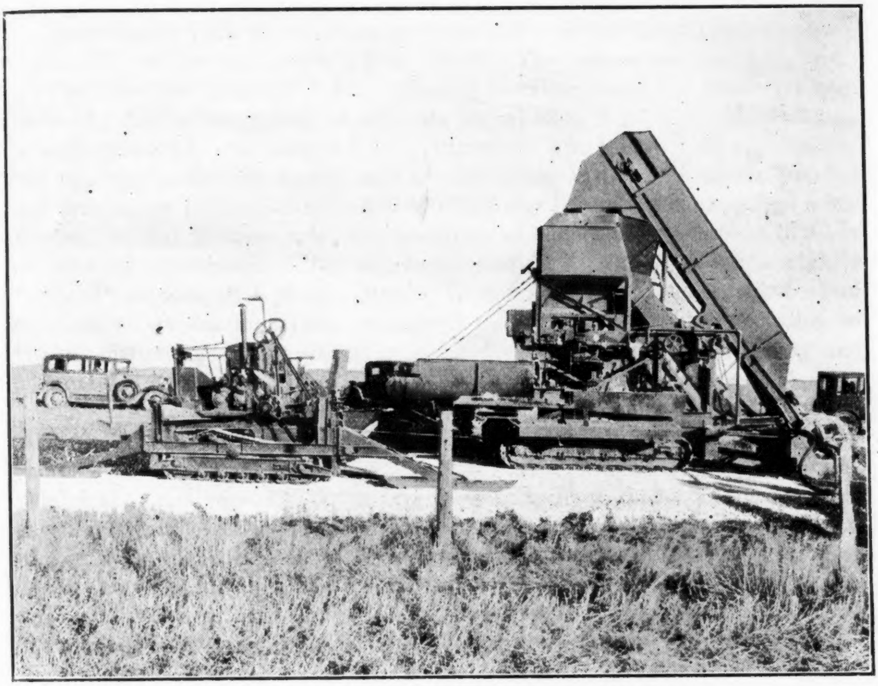

FIG. 3.—A MODERN TYPE OF ROAD-BUILDING EQUIPMENT.

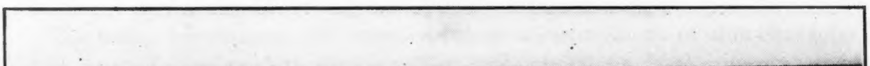

FIG. 4.—PRINCIPLES UPON WHICH A SUB-GRADE EQUALIZER WORKS.

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Immediately back of the spreader trough is a double-faced tamper operating at 200 rev. per min., with a $\frac{3}{4}$-in. stroke. The tamper face is $1\frac{1}{2}$ in. wide, divided into two steps of $\frac{3}{4}$ in., with a $\frac{3}{4}$-in. riser, with the shallower step forward. Immediately back of this is a screed shoe, 6 in. wide, operating on a half-cycle with the tamper. This ingenious arrangement nicely "tucks" the mixture under the screed and provides substantial compaction without any tearing or pulling effect. A 6 000-lb. force is applied directly upon the tamper, so that if those mixtures contain stable cements, including little or no volatiles, subsequent rolling may be eliminated, thus preventing the slightly "rippled" surface that always results from rolling even under the best of conditions. It has long been recognized that direct vertical compression of bituminous paving compositions is ideal if it can be applied uniformly and adequately.

The third outstanding feature of this unit is the application of a system of levers to smooth out humps and depressions in the subgrade. Long skid-shoes with lever-arm attachments to the main frame, extend ahead and behind the crawlers. The lever arrangement is such that a sharp bump of, say, 2-in. height in the subgrade will result in an imperceptible over-all rise of 1 in. in a distance 12 ft. in both directions therefrom. The principle upon which the subgrade equalizer works is demonstrated by Fig. 4 in which, A is a line drawn by straight-edge; B is an exaggerated, rough subgrade; B' is the same roughness transmitted to the pavement by the wheel; and A' shows the roughness removed by means of the equalizer. By this combination of crawlers and subgrade equalizer, the expensive item of forms is eliminated completely and a smooth riding surface is produced.

The many advantages and sound economy of equipment of this character which is really designed to meet varying field conditions, is so apparent as to require little comment. The advantage of plant mix over road mix is two-fold: First, much better coating and superior uniformity of composition; and, second, economy of bitumen by virtue of more complete dispersion. Plant mix saves approximately one-third of the bitumen required for the same aggregate in road mix. This saving alone usually amounts to from \$500 to \$800 per mile of 20-ft. road, 2 to 3 in. thick. When all the merits of plant mixing in large quantities are added to the advantages of mechanical spreading, tamping, and finishing, without the great expense of forms (thus providing equipment that is comfortably capable of mixing and laying a mile or more of 20-ft. road per working day), the attention of the engineer, the builder, and the tax-paying user is quite properly commanded.

APPLICATIONS FOR ADMISSION AND FOR TRANSFER

The Constitution provides that the Board of Direction shall elect or reject all applicants for *Admission* or for *Transfer,* and, in order to determine justly the eligibility of each candidate, the Board must depend largely upon the Membership for information.

This list is issued to members in every grade for the purpose of securing all such available information, and every member is urged to scan carefully each monthly list of candidates and to furnish the Board with data in regard to any applicant which may aid in determining his eligibility. It is the *Duty* of all *Members* to the *Profession* to assist the *Board* in this manner.

It is especially urged, in communications concerning applicants, that errors in the record be pointed out and a *Definite Recommendation as to the Proper Grading in Each Case* be given, inasmuch as the grading must be based upon the opinions of those who know the applicant personally, as well as upon the nature and extent of his professional experience. If facts exist derogatory to the personal character or to the professional reputation of an applicant, they should be promptly communicated to the Board. *Communications Relating to Applicants are considered by the Board as Strictly Confidential.*

The Board of Direction will not consider the applications herein contained from residents of North America until the expiration of thirty (30) days, and from non-residents of North America until the expiration of ninety (90) days from February 15, 1932.

MINIMUM REQUIREMENTS FOR ADMISSION

Grade	General Requirement	Age	Length of Active Practice	Responsible charge of work
Member	Qualified to design as well as to direct work	35 years	12 years*	5 years
Associate Member	Qualified to direct work	27 years	8 years*	1 year
Junior	Qualified for sub-professional work	20 years†	4 years*	
Affiliate	Qualified by scientific acquirements or practical experience to co-operate with engineers	35 years	12 years*	5 years
Fellow	Contributor to the permanent funds of the Society			

* Graduation from a school of engineering of recognized reputation is equivalent to 4 years active practice.

† Membership ceases at age of 33 unless transferred to higher grade.

LIST OF APPLICANTS.

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KEHART, MARTIN W.	Minneapolis, Minn.	
KELLY, IRA D. S.	Topeka, Kans.	
KESTING, BERNARD G.	Toledo, Ohio	
KLOBERG, EDWARD	New York City	
KRUSE, PAUL F.	New York City	
LAIDLAW, DOUGLAS S.	Beauharnois, Que.	
LAWRENCE, HARVEY T.	Mangum, Okla.	
LOTT, JAMES G.	Marfa, Tex.	
MACDONALD, JOHN	Schenectady, N. Y.	
MACKEY, LINCOLN	Washington, D. C.	
MCMAHON, HARRY R.	Clifton, N. J.	
MATSUI, YASUO	New York City	
MAUZY, HARRIS K.	So. Pasadena, Cal.	
MOORE, RAYMOND L.	Toledo, Ohio	
MORRISON, DEMING W.	Sacramento, Cal.	
MOSIER, RAY R.	Paris, Ill.	
MOTA, CANDELARIO C.	Mayaguez, Porto Rico	
NELSON, ARTHUR M.	Bronxville, N. Y.	
O'LEARY, WILLIAM A.	New York City	
PINYAN, RONALD A.	Beaumont, Cal.	
RENZ, EDWIN W.	Philadelphia, Pa.	
SCHUMACHER, KARL F.	San Bernardino, Cal.	
SCHMUCKER, LEROY L.	Akron, Ohio	
SFILIGOJ, BOGOMIR	Ridgefield, N. J.	
SHIELS, THOMAS D.	Abilene, Tex.	
SMIRNOFF, VALENTINE F.	San Francisco, Cal.	
SMITH, DAVID O.	West Chester, Pa.	
SMITH, WALDO E.	Fargo, N. Dak.	
SNYDER, HOWARD H.	New York City	
STARKWEATHER, W. H.	Detroit, Mich.	
THOMSON, GORDON H.	Minneapolis, Minn.	
WALLIN, FRANK A.	Boonton, N. J.	
WAMSLEY, DONALD C.	Dallas, Tex.	
WEINSTOCK, ISIDOR L.	Brooklyn, N. Y.	
YANDA, ALFRED D.	Cleveland, Ohio	
ZISMAN, JOSHUA F.	New York City	

The fact that applicants give the names of certain members as references does not necessarily mean that such members endorse.

The number in the center above each record indicates the serial number of the applicant for the current year, and that at the left the district in which he resides.

The abbreviations in Italics represent respectively, *TT*, Total Time; *SP*, Sub-Professional Work; *P*, Professional Work; *RC*, Responsible Charge; *D*, Design. The figure for Total Time is determined by adding one-half the time spent in Sub-Professional Work to the time spent in Professional Work. The figures showing the amount of time spent in Responsible Charge and on Design are the estimate of the Applicant. The allowance of four years for graduation or of one-half of a year for each academic year successfully completed in an engineering college without graduation is included in Total Time and Professional Work.

FOR ADMISSION

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(1) **ALCAINE, JOSE EUGENIO**, Jr., 270 Broadway, New York City. (Age 38. Born Santa Tecla, El Salvador.) 1910 B. of Sci. and Letters, Instituto Nacional El Salvador. 1920 Ingenieur Arch., Ecole Speciale des Travaux Publics of Paris (France). *TT 4: P 4.*—Jan. 1915 to Jan. 1917 in private practice, on surveys, plans, etc. of properties and bridges. *TT 1: SP 1.*—Feb. 1917 to Feb. 1919 with S. Pearson & Sons, London, England, as Levelman, Transitman and Draftsman, on calculations, plans and studies for municipal works in San Salvador. *TT 2: P 2.*—Nov. 1921 to April 1923 Engr. of Public Works, Republic of Guatemala, and Cons. Engr., on design and construction of roads, bridges, buildings and water supply. *TT 1.5: P 1.5: RC 1.5: D 0.7.*—April 1923 to March 1927 Engr. of roads and bridges, Sec. of Public Works, Republic of El Salvador, on surveys and plans, design and construction of roads and bridges, also Professor of Descriptive Geometry. *TT 4: P 4: RC 4: D 2.*—March 1927 to March 1931 Chf., Sec. of Highways and Bridges, Republic of El Salvador, designed and built roads, bridges and water-works, including pumping plants, pressure mains and distribution for two cities. *TT 4: P 4: RC 4: D 1.*—March to July 1931 Chf., Div. of Water Supply, Republic of El Salvador, in charge of maintenance and operation; Administrative and Technical Inspector of municipal constructions. *TT 0.3: P 0.3: RC 0.3.*—At present Secretary, Consul General of El Salvador, New York City.—*TT 16.8: SP 1: P 15.8: RC 9.8: D 3.7.* Refers to M. E. Gilmore, R. C. Hardman, M. E. Lopez Harrison, C.M. Upham, L. F. Whitbeck.

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(3) **BAIRD, GORDON GIFFORD**, 1661 Steuben St., Utica, N. Y. (Age 26. Born Utica, N. Y.) Licensed Land Surveyor, New York State.—Sept. 1925 to June 1926 student, Union Coll. *TT 0.5: P 0.5.*—Summer 1923 Asst. Asphalt Plant Foreman, Utica Constr. Co., being Inspector and Chemist of Warrenite-bitulithic asphalt pavement and Timekeeper, inspecting concrete mixing plant, street and plant work.—Summer 1922, June to Dec. 1924 and May 1925 to July 1931 (except Sept. 1925 to June 1926) with Div. of Highways, New York State Dept. of Public Works, until Sept. 1925 as Laborer and Inspector, on macadam pavement, concrete culvert construction, testing and inspecting concrete materials, etc., acting as Rodman, Chainman and Instrumentman; June 1926 to Sept. 1928 Jun. Asst. Engr., Grade 1 (Field), surveying, drafting, tracing, earthwork, inspecting highway and bridge and concrete pavement construction, estimating; Sept. 1928 to April 1931 Jun. Asst. Engr., Grade 2 (Field), inspecting sub-base and drainage culverts for macadam road, later Asst. to Project Engr., inspecting macadam road construction, three concrete bridges, in charge of reports, records and made monthly estimates for contract (6 miles), inspecting concrete pavement, in charge of party on right-of-way survey, on design and estimate for new highways, etc.; after April 1931 Asst. Engr., Grade 1, in charge of reconstruction through villages, asphalt pavements, checking pave-

ment mix and controlling uniform plant product, in charge of inspection and survey parties. *TT 3.7: SP 2: P 1.7: RC 0.6: D 0.2.*—July 1931 to date Asst. Engr., Grade 1, Oneida County Dept. of Highways, in charge of 4-mile contract, asphalt road construction, new experimental asphalt surface, inspecting construction, highway surveying; at present designing estimating and general engineering office work. *TT 0.6: P 0.6: RC 0.6.*—*TT 4.8: SP 2: P 2.8: RC 1.2: D 0.2.* Refers to G. R. Bice, L. D. Brownell, D. A. McClung, A. O'Brien, H. V. Owens.

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(10) **BAITY, HERMAN GLENN,** Univ. of North Carolina, Chapel Hill, N. C. (Age 36. Born in Davie County, N. C.) 1917 A. B., and 1922 B. S. in C. E., Univ. of N. C. 1925 M. S. in San. and Mun. Eng., and 1928 Sc. D. in Eng., Harvard Univ. *TT 4: P 4.*—May 1917 to Sept. 1919 with U. S. Army, 3 months as Cadet, Officers' Training Camp, Ft. Oglethorpe, Ga., then 2nd Lieut. and 1st Lieut., Ordnance, in charge of various construction, storage, transportation, issue and repair of ordnance supplies, ammunition and material, railway and depot construction, maintenance and operation and ammunition demolition in U. S. and A. E. F. *TT 2.2: SP 0.2: P 2: RC 1: D 1.*—Summer 1921 Transitman, North Carolina State Highway Comm., Transitman and Draftsman, preparing detail map of Chapel Hill, N. C., and Transitman and Asst. Engr., Univ. of North Carolina, on location and construction of Univ. railroad.—June 1922 to Sept. 1924 Asst. Engr., North Carolina State Board of Health, promotion, review of plans and supervision of operation of State institutional and municipal water supply and treatment, sewerage and sewage-treatment works; collaborating with San. Engr., U. S. Public Health Service on approval of water supplies for interstate carriers. *TT 2.3: P 2.3: RC 2.3: D 2.3.*—Sept. 1924 to Sept. 1926 Research Fellow, The Rockefeller Foundation, research work in sanitary and municipal engineering, Harvard Eng. School and U. S. P. H. S. Stream Investigation Laboratory, studied principal municipal sanitary works and public health organizations in eastern United States conducting group of foreign engineers. *TT 2: P 2: RC 1: D 1.*—Sept. 1926 to date with Univ. of North Carolina, until Sept. 1929 as Associate Prof., and since then Prof., of San. and Mun. Eng., since Sept. 1928 Head of Dept. of Civ. Eng., Feb. to June 1931 Acting Dean, and since June 1931 Dean, of Eng.; also, Sept. 1926 to June 1931 Associate San. Engr., and June 1931 to date Engr. Member, North Carolina State Board of Health, on design and construction of sewage-treatment works in Chapel Hill; Sept. 1926 to date Consultant in stream sanitations, water, sewage and industrial-waste treatment and (since June 1931) supervising operation of water and sewage-treatment plants in Chapel Hill. *TT 5.3: P 5.3: RC 5.3: D 5.3.*—*TT 15.8: SP 0.2: P 15.6: RC 9.6: D 9.6.* Refers to G. M. Fair, T. F. Hickerson, H. E. Miller, W. C. Olsen, W. M. Piatt, T. Saville, H. L. Shaner, C. W. Smedberg.

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(15) **BARNETT, FRANCIS VICTOR,** 707 Jackson St., Monroe, La. Graduating student awarded Mid-South Sec. prize. (Age 23. Born Dermott, Ark.) 1931 B. S. in C. E., Univ. of Ark. *TT 4: P 4.*—June 1927 to June 1928 Instrumentman, Arkansas Power & Light Co. *TT 0.5: SP 0.5.*—June 1931 to date Draftsman, United Gas System, Monroe La.—*TT 0.3: SP 0.3.*—*TT 4.8: SP 0.8: P 4.* Refers to N B. Garver, W. R. Spencer.

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(2) **BIGWOOD, BURKE LINCOLN,** 102 Dover Road, West Hartford, Conn. (Age 35. Born Winooski, Vt.) 1918 B. S. in C. E., Univ. of Vt. *TT 4: P 4.*—June to July 1918 Asst. to City Engr., Burlington, Vt., acting as Instrumentman, Rodman, Inspector, Draftsman, etc. *TT 0.1: SP 0.1.*—Oct. to Dec. 1918 attended Coast Artillery Officers' Training School, Ft. Monroe, Va.; commissioned 2nd Lieut., Reserves. *TT 0.1: P 0.1.*—April 1919 to date with U. S. Geological Survey, as follows: April to Oct. 1919 Field Asst., and Nov. to Dec. 1919 Jun. Engr., Boston, Mass., on stream gauging, 1 month in charge of and responsible for construction of automatic gauge installation; Jan. 1920 to Aug. 1921 Jun. and Asst. Engr., and Dec. 1921 to Jan. 1924 Asst. Engr., Washington, D. C., reviewed and assembled stream-gauging data, tested and studied current meters and other instruments, some special work, etc., Sept. to Nov. 1921 Asst. Engr., Chattanooga, Tenn., on special assignment from Washington office; Feb. 1924 to May 1929 Office Engr., until Sept. 1926 in Honolulu, Hawaii, then in Albany, N. Y., directing office work under supervision of Dist. Engr., and in complete charge of office during his absence; also some field work; June 1929 to date Dist. Engr., Hartford, Conn., in direct charge of all stream-gauging work in Connecticut, being directly responsible in all administrative matters, design, construction, etc. *TT 11.8: SP 0.6: P 11.2: RC 5.3: D 3.2.*—*TT 16: SP 0.7: P 15.3: RC 5.3: D 3.2.* Refers to T. W. Dix, N. C. Grover, A. W. Harrington, A. H. Horton, J. C. Hoyt, C. H. Pierce.

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(2) **EILLMYER, CARROLL DAVIS,** Box 54, Kingston, R. I. (Age 39. Born Shepherdstown, W. Va.) 1914 B. S. in M. E., Va. Pol. Inst. *TT 4: P 4.*—Sept. 1914 to Sept. 1916 Draftsman, Norfolk & Western Ry., Roanoke, Va., on valuation of common carriers, plotting maps, profiles and cross-sections from survey notes. *TT 1: SP 1.*—Sept. 1916 to Aug. 1918 Instructor in Mech. Eng., Throop Coll. of Technology (California Inst. of Technology); also assisted in laboratories. *TT 2: P 2.*—Aug. 1918 to March 1919 with U. S. Army, until Sept. 1918 as Private, then 2nd Lieut., Inf., Personnel Adjutant and later Unit Supply Officer and Unit Finance Officer, S. A. T. C. Throop Coll. of Technology.—April to June 1919 Sales Engr., Worthington Pump & Machinery Corporation, Harrison, N. J.—June 1919 to Aug. 1920 Designer and later Asst. Engr., Atlas Portland Cement Co., Northampton, Pa., field and office work for valuation, N. & B. R. R., estimates and designs for plant extensions and improvements, metallographic and instruction work. *TT 1.2: P 1.2: RC 1: D 0.9.*—Sept. 1920 to June 1924 with Georgia School of Technology, until 1921 as Asst. Prof., then Associate Prof. of mechanics and drawing and machine design, after 1923 also Director of general engineering course, co-operative plan. *TT 3.8: P 3.8: RC 3.8.*—July 1924 to July 1930 with Atlas Portland Cement Co. (Universal-Atlas Cement Co.), until July 1925 as Designer, then Constr. Supt. and after June 1926 Mech. Engr., Power Dept. *TT 6.1: P 6.1: RC 5.1: D 1.*—Aug. 1930 to date Asst. Prof. of Eng., and Supt. of Constr., Rhode Island State Coll., Kingston, R. I., teaching railroad engineering, industrial management, graphic statics and topographic surveying, drawing plans and specifications and supervising construction of improvements and extensions. *TT 1.5: P 1.5: RC 1.5: D 0.5.*—*TT 19.6: SP 1: P 18.6: RC 11.4: D 2.4.* Refers to C. S. Churchill, C. T. Leeds, L. P. Smithey, F. C. Snow, E. Somers, F. Thomas, F. P. Turner.

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(6) **BROWN, ANDREW ALBERT,** 1582 Washington St., Charleston, W. Va. (Age 24. Born New Haven, W. Va.) 1930 B. S. C. E., W. Va. Univ. *TT 4: P 4.*—July 1930 to date Draftsman with L. L. Jemison, Charleston, W. Va., detailing, estimating quantities and some designing of superstructures and substructures of bridges. *TT 0.7: SP 0.7. TT 4.7: SP 0.7: P 4.* Refers to G. P. Boomsliter, R. P. Davis.

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(1) **CAFONE, RALPH GEORGE,** 27 Roma St., Nutley, N. J. (Age 24, Born Nutley, N. J.) 1930 B. S. in C. E. Carnegie Inst. Tech. *TT 4: P 4.*—Jan. 1931 to date Jun. Civ. Engr., New Jersey State Highway Dept., acting as instrumentman and on office work. *TT 0.6: SP 0.6.*—*TT 4.6: SP 0.6: P 4.* Refers to S. T. Barker, J. B. Hayden, D. Ramsay.

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(4) **CAMBLOS, LUCIUS ELMER,** 26 South Harwood Ave., Kirklyn, Pa. (Age 36. Born Philadelphia, Pa.) Sept. 1914 to June 1915 student, Drexel Inst. *TT 0.5: P 0.5.*—July 1915 to July 1918 with F. H. Clement & Co., Gen. Constrs., Philadelphia and Bethlehem, Pa., until April 1916 as Rodman and Timekeeper, then Asst. to Chf. Engr., supervising reinforced concrete work, including heavy foundations, retaining walls, ore docks and cableways, also railroad construction, comprising steam-shovel work, fills, bridge work, sewers, etc., latter part of time in direct charge of several jobs. *TT 1.8: SP 1.3: P 0.5.*—July 1918 to Sept. 1919 with U. S. Marine Corps, A.E.F., France.—Oct. 1919 to date with A. Raymond Raff Co., Philadelphia, Pa., until Dec. 1920 as Eng. Draftsman and Asst. to Engr., preparing scale drawings and specifications and assisting in checking computations for structural designs, and since Jan. 1921 Engr., designed and had entire charge of construction on various classes of timber, structural steel and reinforced concrete structures, including factory and warehouse buildings, power and boiler house, reinforced concrete foundations, machine shops, etc.; also prepared engineering reports and appraisals (given in detail in application). *TT 12: SP 0.4: P 11.6: RC 11.1: D 8.*—*TT 14.3: SP 1.7: P 12.6: RC 11.1: D 8.* Refers to J. Cantley, B. Lafferty, M. D. Scott, E. E. Seyfert, C. M. Stewart.

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(4) **CARROLL, WILLIAM JOSEPH,** 422 East Hector St., Conshohocken, Pa. (Age 34. Born Conshohocken, Pa.) 1923 B. S. in C. E., Drexel Inst. *TT 4: P 4.*—Sept. 1917 to Sept. 1918 Rodman, Chainman and Instrumentman with James Cresson, Civ. Engr., Norristown, Pa., and Engr., Montgomery Co., Pennsylvania, highway and bridge surveys, subdivisions, etc. *TT 0.5: SP 0.5.*—July 1923 to Feb. 1924 Field Engr. with Irwin & Leighton, Contrs., Philadelphia, lines, grades, checking plans and submitting weekly cost report on Girard Coll. Armory. *TT 0.6: P 0.6: RC 0.6.*—Feb. 1924 to Oct. 1925 Asst.

Highway Engr., Burlington County, N. J., on design and supervision of construction of highway bridges and of penetration macadam, sheet asphalt and concrete roads. *TT 1.6: P 1.6: RC 1.6: D 1.6.*—Oct. 1925 to Jan. 1927 and March to Nov. 1927 Constr. Supt. for Staub & Kolyn, Engrs. and Contrs., Trenton, N. J., on construction of concrete highway bridges *TT 1.9: P 1.9: RC 1.9.*—Jan. to March 1927 Estimator and Detailer, The Labar Steel Co., Philadelphia, Pa., on reinforcing steel for various building projects. *TT 0.2: P 0.2: RC 0.2: D 0.2.*—Jan to April 1928 Instrumentman with Battey & Kipp, Contrs., Chicago, Ill., on Campbell Soup Co. Bldg., Camden, N. J., lines, grades and layout of railroad yard. *TT 0.1: SP 0.1.*—Sept. 1928 to June 1930 Engr. in charge of all operations for Smith & Branin, Mun. and San. Engrs., Mount Holly, N. J., subdivisions, topographical surveys, design and supervision of construction of streets, highways, bridges, storm-water drainage. *TT 1.8: P 1.8: RC 1.8: D 1.8.*—June 1930 to date Div. Engr., The Bituminous Service Co., Inc., West Chester, Pa., estimating, designing and supervising construction of bituminous macadam roads. *TT 1.6: P 1.6: RC 1.6: D 1.6.*—*TT 12.3: SP 0.6: P 11.7: RC 7.7: D 5.2.* Refers to F. L. Branin, W. Easby, Jr., M. Goodkind, M. den H. Kolyn, R. Radbill, H. B. Smith

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(15) **CARTER, RUFUS HENRY, Jr.**, 223 Santa Fe Ave., Santa Fe, N. Mex. (Age 22. Born Raton, N. Mex.) 1931 B. S. in C. E., Univ. of N. Mex. *TT 4: P 4.*—Oct. to Nov. 1931 Chainman with City Engr., Santa Fe, N. Mex.; June to Sept. 1931 Transitman, and Dec. 1931 to date Draftsman, New Mexico State Highway Dept. *TT 0.3: SP 0.3.*—*TT 4.3: SP 0.3: P 4.* Refers to J. H. Dorroh, P. S. Fox, G. M. Neel.

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(1) **CHAMBERLIN, CLARENCE VAN CLINTON**, Shrewsbury Ave., Red Bank, N. J. (Age 34. Born La Porte City, Iowa.) 1923 B. S., Mass. Inst. Tech. *TT 4: P 4.*—Sept. 1916 to June 1919 student, until May 1917 at Carleton Coll., then successively at U. S. Navy Elec., Navigation and Steam Eng. Schools.—Summers 1920 and 1921 and June 1923 to June 1926 with Turner Constr. Co., first as Draftsman and Material Clerk, June to Dec. 1923 Engr. on reinforced concrete power house ($20 000) and factory building ($800 000), having charge of layout, line and grade, being Asst. Supt. in charge of coffer-dam and head-gate construction, laying cast-iron water-mains, etc.; Dec. 1923 to April 1925 Asst. Supt. on three reinforced concrete factories ($1 800 000), having charge of reinforcing steel, structural steel erection, caisson foundations, etc.; Jan. to March 1926 Expeditor on high-class office building, scheduling and delivering sub-contract material; April to Dec. 1925 and March to June 1926 Supt., in charge of construction of a reinforced concrete factory ($350 000) and in charge of improvement of grounds, building roads, drainage system, erecting flood-light towers, running track, installing granite curb (1 mile) for stadium ($750 000). *TT 3: P 3: RC 1.5.*—July 1926 to Nov. 1930 and May 1931 to date Supt. in charge on hospital construction (total $3 625 000), until April 1927 with Feeney & Sheehan, Albany, N. Y., May 1927 to Nov. 1930 with Hugh Montague, Inc., and since May 1931 with Edw. M. Waldron, Inc. *TT 5.1: P 5.1: RC 5.1.*—*TT 12.1: P 12.1: RC 6.6.* Refers to D. H. Dixon, L. H. Doane, C. W. Ryan, C. H. Schwertner, L. F. Wright.

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(1) **COLLINS, RICHARD NICHOLAS**, 60 John St., New York City. (Age 35. Born Wilkes-Barre, Pa.) 1924 B. S. in Civ. Eng., Towne Sci. School, Univ. of Pa. *TT 4: P 4.*—Aug. 1914 to Sept. 1915 Recorder, Lehigh Valley Coal Co. and Sept. 1915 to Dec. 1917 Noteman and Transitman, Delaware and Hudson Coal Co., mine surveys, mapping and plotting. *TT 1.6: SP 1.6.*—Dec. 1917 to July 1919 Sergeant of Engrs., U. S. Army, building wooden trestle and pontoon bridges. *TT 0.8: SP 0.8.*—Sept. 1924 to Oct. 1928 with Lehigh Valley R. R., until June 1926 as Transitman on design, and field layout of tracks, yards, piers and bridges, then Designing Draftsman, on design and checking design of truss and girder bridges, estimating cost, and checking fabricators shop drawings. *TT 4: P 4: RC 2.8: D 2.2.*—Nov. 1928 to July 1930 Concrete and Structural Steel Designer, United Engrs. & Constrs., on design and checking design and fabricators shop drawings of subways, office buildings and railroad bridges, for Philadelphia Improvements of Pennsylvania R.R. *TT 1.7: P 1.7: RC 1.7: D 1.7.*—Aug. 1930 to March 1931 Concrete Designer, Anglo-Chilean Nitrate Corporation, on design of industrial buildings, preparation of detail concrete drawings, Pecko de Valdiva Plant erected in Chile. *TT 0.6: P 0.6: RC 0.6: D 0.3.*—April 1931 to date Asst. Engr., New York Central R.R., on design of concrete and steel viaduct for elevated highway over N. Y. C. tracks in New York City. *TT 0.8: P 0.8: RC 0.8: D 0.8.*—*TT 13.4: SP 2.4: P 11: RC 5.8: D 5.* Refers to R. Kaysser, A. H. Reeves, H. T. Rights, F. C. Stehle, H. A. Wistrich.

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(9) **CRAMPTON, LAURENCE HARLOW,** Murlin Heights, R. F. D. 1, Dayton, Ohio. (Age 32. Born in Butler Township, Ohio.) May to Sept. 1919 and June 1920 to Sept. 1925 with R. P. Sebold, Dayton, Ohio, on municipal engineering and surveying, after June 1920 being Associate Engr. in charge of sewer construction, small bridges, buildings, foundations, excavations, subdividing property, surveying, drainage, etc. *TT 5.4: SP 0.1: P 5.3: RC 5.3: D 5.3.*—Sept. 1919 to June 1920 with Miami Conservancy Dist., in field party on construction at Taylorsville Dam. *TT 0.4: SP 0.4.*—Sept. 1925 to date Engr.-Contr., Dayton; on road and street, sewer, drainage and brick, asphalt and macadam pavement construction; designed and installed monolithic concrete, precast concrete and vitrified sewers, cast-iron water mains, lead water pipe, sewage-disposal plants, fish ponds, swimming pools, stone masonry, etc., for Montgomery County, City of Dayton, Village of Oakwood, Ohio Dept. of Highways, etc.; also did surveying and excavating. *TT 6.3: P 6.3: RC 6.3: D 6.3.*—*TT 12.1: SP 0.5: P 11.6: RC 11.6: D 11.6.* Refers to K. B. Allen, G. F. Baker, F. J. Cellarius, J. F. Hale, C. H. Paul, W. O. Pease.

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(7) **DHILLON, ARJAN SINGH,** 332 East Madison St., Ann Arbor, Mich. (Age 24. Born Harikey, Punjab, India.) 1931 B. S. E., Univ. of Mich. *TT 4: P 4.* Refers to J. H. Cissel, H. W. King, R. L. Morrison, R. H. Sherlock, C. O. Wisler, J. S. Worley.

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(2) **DORRANCE, WILLIAM TULLY, Jr.,** 103 Armory St., New Haven, Conn. (Age 24. Born White Plains, N. Y.) 1932 C. E., Brooklyn Pol. Inst. *TT 4: P 4.*—Summers Timekeeper, C. W. Blakeslee and Sons, New Haven, Conn. (1927), Eng. Laborer, acting as Rodman and Instrumentman, Westchester County Park Comm., Bronxville, N. Y. (1929), Rodman and Instrumentman, Student Eng. Camp, Delaware and Hudson R. R. Corporation, Plattsburg, N. Y. (1930).—*TT 4: P 4.* Refers to H. R. Buck, H. R. Codwise, H. P. Hammond, L. G. Holleran, L. F. Rader, E. J. Squire.

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(1) **FENERDJIAN, HRANT ARMENAG,** Argous 5, Athens, Greece. (Age 26. Born Marsovan, Turkey.) 1927 B. S. C. E., Robert Coll., Constantinople, Turkey. *TT 4: P 4.*—Sept. 1927 to date with Ulen & Co., Athens, Greece, until March 1929 as Draftsman, then Chf. Draftsman, Athens Water Supply. *TT 4.4: P 4.4: D 4.4*—*TT 8.4: P 8.4: D. 4.4.* Refers to A. B. Christensen, V. M. Crown, R. W. Gausmann, R. H. Keays, R. M. Merriman, C. H. Sutherland.

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(13) **FOSTER, HERBERT BISMARCK, Jr.,** 834 Mendocino Ave., Berkeley, Cal. (Age 23. Born Berkeley, Cal.) 1931 B. S., Univ of Cal. *TT 4: P 4.*—Oct. 1931 to date Jun. San. Engr., California State Dept. of Public Health. *TT 0.1: SP 0.1.*—*TT 4.1: SP 0.1: P 4.* Refers to C. Derleth, Jr., H. D. Dewell, B. A. Etcheverry, F. S. Foote, B. C. Gerwick, C. G. Gillespie, C. G. Hyde, E. A. Reinke, H. C. Vensano.

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(8) **FULTON, EDWARD ARTHUR,** 701 North Michigan Ave., Chicago, Ill. (Age 33. Born Parrsboro, N. S., Canada.) 1924 B. S. in C. E., Univ. of Manitoba. 1930 M. S., Mass. Inst. Tech. *TT 4: P 4.*—Summer 1920 Instrumentman, and June 1921 to Oct. 1922 Instrumentman and Draftsman, Dept of Interior, Reclamation Service of Canada, on location and construction of irrigation works. *TT 0.7: SP 0.7.*—Summer 1923 Instrumentman, Illinois Central R. R., June 1924 to Jan. 1925 Draftsman and Designer with Pearse, Greeley & Hansen, Cons. Engrs., Chicago, on water-purification and sewage disposal plants. *TT 0.7: P 0.7: RC 0.7: D 0.7.*—Jan. to Sept. 1925 Res. Engr. with Consoer, Older & Quinlan, Chicago, on sewer construction, paving and water-works. *TT 0.7: P 0.7: RC 0.7: D 0.7.*—Feb. 1926 to Aug. 1927 Asst. Engr. with Solomon, Norcross & Keis, Cons. Engrs., Ft. Lauderdale, Fla., on design and construction of water-supply and purification, sewers, sewage-disposal and other municipal improvements. *TT 1.5: P 1.5: RC 1.5: D 1.5.*—Aug. 1927 to Feb. 1930 Asst. Engr. with Wiedeman & Singleton, Cons. Engrs., Atlanta, Ga., on design, construction, valuations, investigations, reports, etc., on water supplies and purification of sewerage and sewage disposal, water power and storage, hydraulic mining projects, etc. *TT 2.5: P 2.5: RC 2.5: D 2.5.*—June 1930 to date Sales Engr., The Dorr Co., New York City. *TT 1.7: P 1.7: RC 1.7: D 1.7.*—*TT 11.8: SP 0.7: P 11.1: RC 7.1: D 7.1.* Refers to H. K. Barrows, J. N. Finlayson, F. J. Keis, G. R. Solomon, H. F. Wiedeman.

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(2) **FURNESS, LAWTON WILLIAMS,** State House, Augusta, Me. (Age 25. Born Stafford, Conn.) 1928 B.Sc., Carnegie Inst. Tech. *TT 4: P 4.*—May 1928 to May 1929 Draftsman, Blaw Knox Co., Blawnox, Pa., tracing and detailing steel towers and steel buildings. *TT 0.5: SP 0.5.*—May 1929 to date Jun. Hydr. Engr., Water Resources Branch, U. S. Geological Survey, stream gauging, investigation, construction and maintenance of gauging stations, compilation and computation of stream flow data. *TT 2: SP 0.7: P 1.3: RC 0.5.*—*TT 6.5: SP 1.2: P 5.3: RC 0.5.* Refers to F. M. McCullough, M. R. Stackpole.

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(16) **GIBSON, WILLIAM EVERETT,** 219 North Sixth St., Manhattan, Kans. (Age 30, Born Arrington, Kans.) 1927 B. S. in C. E., Kans. State Coll. of Agri. and Applied Sci. *TT 4: P 4.*—July 1927 to May 1930 Materials Inspector, inspecting road materials, and May 1930 to date Engr. of Tests, Kansas State Highway Comm., from Nov. 1927 to May 1928 being Asst. to Inspector in charge, then in charge of branch Laboratory, and since May 1930 in charge of Road Materials Laboratory, Manhattan, Kans. *TT 4.2: SP 0.1: P 4.1: RC 3: D 0.6*—*TT 8.2: SP 0.1: P 8.1: RC 3: D 0.6.* Refers to H. Allen, H. D. Barnes, W. V. Buck, L. E. Conrad, E. R. Dawley, M. W. Furr, R. H. Pennartz, C. H. Scholer, I. E. Taylor, O. K. Williamson.

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(16) **GILLMAN, HAROLD LEETON,** Meade, Kans. (Age 29. Born New Cambria, Kans.) 1926 B. S. in Civ. Eng., Kans. State Coll. of Agri. & Applied Sci. *TT 4: P 4.*—June to Sept. 1926 Asst. County Engr., Osage County, being Inspector and Instrumentman. *TT 0.1: SP 0.1.*—Sept. 1926 to date County Engr., Meade County, Kans., in charge of construction and design of roads and bridges, except on State System of Roads; Oct. 1927 to June 1928 also Res. Engr., Kansas State Highway Comm., in charge of construction of bridge and short road project on State Highway System. *TT 5.3: P 5.3: RC 5.3: D 5.3.*—*TT 9.4: SP 0.1: P 9.3: RC 5.3: D 5.3.* Refers to L. E. Conrad, R. C. Ham, C. E. Hommon, J. A. Roby, M. A. Wilson.

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(10) **HANNOCK, CHARLES GUSTAV,** P. O. Box 1618, Miami, Fla. (Age 51. Born Auburn, N. Y.) Registered Prof. Engr., State of Florida—Student in Eng., Cornell Univ. (Sept. 1898 to Dec. 1899, Sept. 1900 to June 1901 and Feb. to June 1902) and Rensselaer Polytechnic Inst. (Sept. 1902 to June 1906) *TT 3: P 3*—Aug. to Oct. 1906 Rodman, and Oct. 1906 to Feb. 1907 Asst. Engr., Jamaica Improvement Comm. *TT 0.4: SP 0.1: P 0.3: RC 0.3*—Feb. 1907 to Nov. 1914 Topographical Draftsman, 7 months with Richmond Borough, then with Bureau of Substructures, Brooklyn (4½ years) and Manhattan Borough (2 3/5 years), New York City. *TT 5.1: SP 2.5: P 2.6: RC 2.6*—Nov. 1914 to date in private practice as Chas. G. Hannock, Engr., designed timber bulkhead and fill and designed and installed surface and part of subsurface construction (April 1915 to Jan. 1916), Supt. of Constr. on Ocean Boulevard, Miami Beach, Fla. (Oct. 1917 to March 1918) Engr., in charge as civilian, of construction at Chapman Field, Miami and later at Aviation Gen. Supply depot at Middletown, Pa. (April 1918 to Feb. 1919), Engr. for various Atlantic Shores, Broward Co., Fla., Picture City, Palm Beach, Fla., The Altos de Mars, Miami Beach, and other developments (Jan. 1921 to June 1926), and general engineering; since June 1929 Engr. for Ocean Beach Heights development in Miami Beach (expenditure to date over $1 000 000) also Feb. 1919 to Jan. 1925 member of firm Hannock & Berryman, Contrs., and since Jan. 1929 Superv. Engr. for Dade County, Fla. *TT 17.2: P 17.2: RC 15: D 15.*—*TT 25.7: SP 2.6: P 23.1: RC 17.9: D 15.* Refers to E. Friedman, M. N. Lipp, G. P. Morrill, E. R. Neff, C. S. Nichols, R. W. Reed, O. A. Sandquist.

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(11) **HAPGOOD, EUGENE PALMER,** 714 North Philadelphia St., Anaheim, Cal. (Age 51. Born Cleveland, Ohio.) 1902 B. Sc. (Met.), Ohio State Univ. *TT 4: P 4.*—July 1902 to Feb. 1903 Engr. and Supt., Imperial Clay Co., New Lexington, Ohio, ceramic plant construction and operation. *TT 0.6: SP 0.4: P 0.4: RC 0.4: D 0.4.*—April to Dec. 1903 Chf. of Party, Copper Belt Ry., Bingham Canyon, Utah, design and construction of trestles and retaining walls, etc. *TT 0.6: SP 0.1: P 0.5: RC 0.5: D 0.2.*—Feb. 1904 to March 1907 and April 1910 to Aug. 1911 Chf. of Party and Asst. Engr., Los Angeles & Salt Lake R. R., location, construction and maintenance, in charge of field parties on reconstruction, involving bank widening, ballasting, laying heavy steel, tunnel enlargement, fitting spirals to curves, water-supply lines; also design (except standard structures) and supervision

of construction of terminal facilities at Caliente, Nev., Midford and Lynndyl, Utah. *TT 4.1: SP 0.3: P 3.8: RC 3.8: D 1.6.*—March to Oct. 1907 Engr., Ohio Copper Co., Lark, Utah, in charge of concentrator construction, on design and construction of foundations, retaining walls, trestles, reservoir, pumping plant, houses, shops, etc., supervising erection of steel mill building and machinery installation. *TT 0.6: P 0.6: RC 0.6: D 0.2.*—July 1908 to Feb. 1909 City Engr., Belgrade, Mont., establishing street grades, constructing sidewalks, curbs, etc. *TT 0.6: P 0.6: RC 0.6: D 0.2.*—Feb. to Sept. 1909 in private practice, Harlowton, Mont., surveys and plans for irrigation systems, engineering report on proposed 35-mile railroad between Harlowton and Melville, Mont., including preliminary location and topographic survey, paper location, estimates, etc. *TT 0.6: P 0.6: RC 0.6: D 0.3.*—Oct. 1909 to April 1910 and Sept. 1911 to July 1912 Asst. Engr., City Engr.'s Office, Salt Lake City, Utah, on studies and plans for street improvements, sewers, water supply, reservoirs and dams. *TT 0.9: SP 0.4: P 0.5: RC 0.4: D 0.2.*—July 1912 to May 1915 with Utah Railway Co., Salt Lake City, 6 months as Office Engr., in charge of design, construction specifications and contracts, etc.; 2 months Right-of-Way Agt., 1 year Engr. in charge of construction of North End (20 miles), involving tunnels, moving highways, high-tension power and telephone toll lines, irrigation canals, a river and buildings and protecting parallel railroad, 3 months Consultant on South End (20 miles), relocation surveys on stream crossings, investigating bridge sites, strengthening tunnel linings and reclassifying excavation and borrow pits on entire line; 10 months in Chf. Engr's Office accounting, purchasing, making I. C. C. valuation maps, etc. *TT 2.8: P 2.8: RC 2.8: D 2.*—June to Aug. 1915 Engr., United States Smelting Co., Salt Lake City, in charge of plans for remodeling and enlarging concentrator, machinery layouts, building construction. *TT 0.2: P 0.2: RC 0.2: D 0.2.*—Sept. to Oct. 1915 Engr., Carbon Fuel Co and Salt Lake & Utah Ry. Co., in charge of railroad location in Utah. *TT 0.2: P 0.2: RC 0.2.*—June to Sept. 1916 Engr., Big Four Exploration Co., Park City, Utah, in charge of machinery layout and installation, development of water supply, including reservoir and pumping plant, construction of railroad trestles, etc., operating concentrator. *TT 0.3: P 0.3: RC 0.3: D 0.3.*—Oct. to Nov. 1916 and May 1918 to April 1920 Engr., Gen. Eng. Co., Salt Lake City, on designs of concentrator plants, mill machinery design, manufacture, sales, etc., acting as Purchasing Agt. and Traffic Mgr., handling shipments of ore samples, exporting mill machinery, etc. *TT 2.2: P 2.2: RC 2.2: D 2.2.*—Dec. 1916 to Jan. 1918 Engr., Lynch-Cannon Eng. Co., Salt Lake City, moving sugar factory from Raymond, Alberta, Canada, to Cornish, Utah, involving measuring and numbering structural steel and machinery, designing steel framework of new building to utilize old steel, designing beet sheds and trestles, foundations for buildings, structures and machinery, powerplant and machinery layouts, water-supply and pipe lines, Asst. Supt. of construction and test operation, etc. *TT 1.1: P 1.1.: RC 1.1: D 1.1.*—Feb. to May 1918 Engr. with O. B. Hofstrand, Cons. Engr., Salt Lake City, design of concentrator plant, concrete foundations, retaining walls, ore bins, structural steel frames for mill buildings and machinery layouts. *TT 0.3: P 0.3: RC 0.3: D 0.3.*—May 1920 to Sept. 1921 Cons. Engr. with Huddleson & Fiero, Cons. Engrs., Salt Lake City, on plans, specifications, contracts, construction, etc. for sewer and water-supply systems and reservoirs, sidewalks, curbs, waterways and paving for various municipalities in Utah and Idaho, water-right surveys, etc. *TT 1.4: P 1.4: RC 1.4: D 0.4.*—Sept. 1921-May 1924 Structural Engr. and Superv. Archt. with M. Eugene Durfee, Archt., Anaheim and Fullerton, Cal., on design of foundations, underpinning of buildings and cantilever footings, concrete and steel work, supervising construction, etc., of various buildings in Fullerton and Anaheim, Cal. (some named in application); work also included plans, specifications, contracts and supervision of all buildings and structures in Anaheim City Park, with filtering, heating and recirculating system for plunge. *TT 2.7: P 2.7: RC 2: D 2.7.*—June 1924 to date City Engr., Anaheim, in charge of street openings, widening and improvements and construction of public works and municipal buildings and structures, city planning and zoning, maintenance work on joint outfall sewer, etc.; also private practice as Consultant on concrete and structural work and street proceedings. *TT 7.5: P 7.5: RC 7.5: D 7.5*—*TT 30.8: SP 1.1: P 29.7: RC 24.9: D 19.8.* Refers to W. R. Armstrong, P. Bailey, A. F. Barnard, W. W. Hoy, H. S. Kerr, J. H. Knowles, N. H. Neff, C. A. Smith, M. N. Thompson.

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(8) **HOWARD, WILLIAM RAPPE,** 400 West Madison St., Chicago, Ill. (Age 39. Born at St. Paul, Minn.) 1916 C. E., Univ. of Cin. *TT 4: P 4.*—July 1916 to April 1918 Constr. Supt., Ferro Concrete Constr. Co., Cincinnati, Ohio, in charge of constructing two warehouse buildings, a boiler-house substructure and a factory building. *TT 1.7: P 1.7: RC 1.7: D 1.7*—May 1918 to May 1919 with U. S. Army as Private and Corporal, 308th

Engrs., Sergeant and Lieut., Army Service Corps, G. H. Q., Chaumont, France.—Sept. 1919 to Dec. 1920 Designing and Constr. Engr., Paul Delaney Food Products Co., Brocton, N. Y., designed and constructed concrete unloading platform, concrete warehouse (1 story and basement, tile walls, wood roof trusses) with cold-storage room in basement, enlarged boiler house, installing 300-h. p. boiler and erecting 125-ft. brick stack, revised bottling, capping and conveying equipment. *TT 1.3: P 1.3: RC 1.3: D 1.3.*—April to Dec. 1921 Asst. to Engr. of Inspection and Tests, Highway Dept. of Minnesota, St. Paul, investigated and reported on gravel deposits throughout Minnesota. *TT 0.7: P 0.7: RC 0.7: D 0.7.*—Jan. to March 1922 Appraisal Engr., Eng. Appraisal Co., Minneapolis, Minn., appraised coal yards and grain elevator. *TT 0.3: P 0.3.*—April 1922 to date with A. Guthrie & Co., Inc., St. Paul, Minn., as Office Engr. on design of plant equipment for New Cascade Tunnel, heating plant for shop and camp buildings, iron ore stripping operations, Calumet, Minn., alterations and repairs to equipment yard and shop buildings in Gloster, Minn.; about 2 years Estimator, estimated Stadium Univ. of Minn., Robert St. Bridge ($1 600 000), Union Depot, Water-Works Aqueduct ($50 000), Ford Assembling Plant, all in St. Paul, Cedar Ave. Bridge ($800 000), Minneapolis, Minn., Northern Pacific Coal Stripping, Montana and many railroad and highway bridges; Constr. Supt. on Northern Pacific R. R. grade separation in Minneapolis ($100 000), yard for Pennsylvania R. R. in Columbus, Ohio ($100 000), five grade separations, grading (6 miles) and track laying (6 miles) for Detroit, Toledo & Ironton R. R., Mineral Ridge Dam in Youngstown, Ohio and 18 grade separations for Grand Trunk R. R. in Detroit. *TT 9.7: P 9.7: RC 5.7: D 7.7.*—*TT 17.7: P 17.7: RC 9.4: D 11.4.* Refers to J. C. Baxter, E H. Bruntlett, C. B. Cornell, R. M. Knox, L. E. Ott, H. Schneider.

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(10) JONES, GILES PAUL, Box 62, Macon, Ga. (Age 32. Born Macon, Ga.) 1921 B. S. in Mech. Eng., Ga. School Tech. *TT 4: P 4.*—July 1921 to date with Cornell-Young Co., Inc., Macon, Ga., as follows: July 1921 to Feb. 1924 Timekeeper on culvert and bridge construction for State of North Carolina and construction of fill over Santee River, grading, for State of South Carolina, etc.; Feb. to Dec. 1924 Supt. in charge of contract with Atlantic Coast Line R. R. for construction of approx. 27 miles of second track, Charleston to Jacksonboro, S. C., consisting of grading, track laying and surfacing (approx. $207 000); since Dec. 1924 Vice-Pres., until June 1930 in direct charge of contracts with Atlantic Coast Line R. R. for Tampa concrete underpass, track construction, grading, grade revision, at various places (total approx. $560 000, given in detail in application), contracts with Southern Sugar Co. for construction of grading and tracks at Clewiston and Canal Point, Fla. ($73 000) and contracts with Florida East Coast R. R. for grading, pile trestles, steel bridges and track ($208 000) for Okeechobee Extension, and others; since June 1930 organized paving outfit and had charge of grading and concrete paving contracts in North and South Carolina (total $849 000). *TT 9.2: SP 1.3: P 7.9: RC 7.9: D 4.6*—*TT 13.2: SP 1.3: P 11.9: RC 7.9: D 4.6.* Refers to S. C. Dreyfus, W. Mahone, Jr., J. L. Parker, R. Y. Patterson, W. A. Young.

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(1) KAUFMAN, HARRY, 18 Hegeman Ave., Brooklyn, N. Y. (Age 39. Born Brooklyn, N. Y.) 1922 C. E., International Correspondence Schools—April 1914 to Jan. 1916 Chainman and Jan. 1916 to June 1917 Jun. Asst. Engr., New York State Dept. of Highways, Rochester Div., and later Public Service Comm., New York. *TT 1.6: SP 1.6.*—Aug. 1917 to Feb. 1919 Draftsman, The Foundation Co., New York City. *TT 0.7: SP 0.7.*—July 1919 to Jan. 1923 Draftsman, Dover Boiler Works, New York City, detailing steel plate work. *TT 1.7: SP 1.7*—Jan. to July 1923 Steel (structural) Detailer, Fagan Iron Works, Jersey City, N. J. *TT 0.3: SP 0.3.*—July 1923 to Sept. 1927 Steel (structural) Detailer and Checker, until Nov. 1924 with Palmer Steel Co., Springfield, Mass., then with Levering & Garrigues Co., New York City, and after Aug. 1926 with Taylor Fichter Steel Constr. Co., New York City, checking drawings. *TT 4.2: P 4.2.*—Sept. 1927 to Feb. 1928 Designing Engr., Engineering Service Co., on steel structures. *TT 0.4: P 0.4: D 0.4.*—Feb. 1928 to July 1929 Engr., Atlantic Gypsum Products Co., New York City, on design and supervision of erection of new plant. *TT 1.4: P 1.4: RC 0.4: D 1.*—July 1929 to Sept. 1930 Designing Engr., Delaware, Lackawanna & Western R. R., Hoboken, N. J., design and details of structural steel, bridges, buildings, etc. *TT 1.2: P 1.2: D 1.2.*—Jan. to Sept. 1931 Engr., Furman Eng. Co., New York City, on layout of plant machinery and equipment for same. *TT 0.7: P 0.7: RC 0.7: D 0.7.*—*TT 12.1: SP 4.3: P 7.8: RC 1.1: D 3.2.* Refers to A. B. Cohen, M. Hirschthal, C. M. Segraves, J. L. Vogel, A. C. Waghorne.

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(7) KEHART, MARTIN WILLIAM, 2009 Garfield Ave., Minneapolis, Minn. (Age 29. Born Rathmel, Pa.) 1926 B. Sc. in Civ. Eng., Univ. of Ill. *TT 4: P 4.*—June to Sept. 1925 and June 1926 to March 1927 Surveyman, Corps of Engrs., U. S. Army, surveying and supervising dredging in maintenance of Illinois River, Ill., also (3 months) drafting. *TT 1: P 1.*—April 1927 to Jan. 1928 Draftsman (structural) and Designer, Arkansas Highway Bridge Dept., Little Rock, Ark. *TT 0.8: P 0.8: D 0.8.*—Jan. 1928 to date with Lakeside Bridge & Steel Co., Milwaukee, Wis., as follows: Jan. to Dec. 1928 Constr. Engr., on Dardanell Bridge (concrete and steel) across Arkansas River ($600 000); Dec. 1928–March 1929 Res. Engr. and inspector on steel erection, Univ. of Chicago power house, Chicago, Ill.; April to Oct. 1929 Field Engr., bidding on new bridges, and in charge of construction of concrete and steel bridges; Oct. to Dec. 1929 Steel Supt. and Engr. on highway bridge ($100 000) across Saline River at Warren, Ark., also Asst. to Res. Engr., Steel Supt. and Reinforcing Steel Foreman; Dec. 1929 to September 1930 Constr. Engr., on highway bridge (¾ mile, $348 000) across South Canadian River at Cheyenne, Okla., being in charge of field office; Sept. to Nov. 1930 in charge of preliminary survey for proposed railroad bridge across Manistee River at Manistee, Mich., made complete survey, investigation and estimate; Nov. 1930 to Jan. 1931 Engr. on building construction, including erecting steel for A. O. Smith Building in Milwaukee, Wis.; Jan. to Nov. 1931 Res. Engr. in complete charge of constructing concrete and steel bridge ($90 000) across Barataria Bayou, at Wagners Ferry, Jefferson Parish, La.; Dec. 1931 to date Field Engr. on erection of dam gates and valve gates on government dam across Mississippi River at Minneapolis, Minn. *TT 4: P 4: RC 3.9: D 0.1.*—*TT 9.8: P 9.8: RC 3.9: D 0.9.* Refers to H. Cross, N. B. Garver, F. A. Gerig, R. C. Gibson, B. E. W. Stout.

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(9) KESTING, BERNARD GEORGE, 173 East Broadway, Toledo, Ohio. (Age 30. Born Toledo, Ohio.) 1925 B. S. in C. E., Univ. of Notre Dame. *TT 4: P 4.*—July 1925 to March 1926 Inspector and Draftsman, Pennsylvania State Highway Dept. *TT 0.4: SP 0.4.*—March 1926 to July 1929 Asst. Engr. (13 months) and Asst. Div. Engr., Ohio State Highway Dept., checking plans and estimates for and supervising construction of highways and bridges; superintended construction of addition to Div. Garage. *TT 2.7: SP 0.5: P 2.2: RC 0.2.*—July to Nov. 1929 Engr., A. Bentley & Sons Co., Toledo, acting as Instrumentman on building construction. *TT 0.2: SP 0.2.*—Feb. to Sept. 1930 Engr., J. H. Berkebile & Sons, Toledo, in charge of layout and Asst. to Supt. of Constr.—*TT 0.7: P 0.7: RC 0.7.*—Sept. 1930 to Jan. 1931 Engr., Spicer Mfg. Corporation, Toledo, inspecting construction of new factory unit. *TT 0.3: P 0.3.*—April 1931 to date Inspector (4 months) and Asst. Engr., Ohio State Highway Dept., drawing highway and bridge plans. *TT 0.5: SP 0.2: P 0.3.*—*TT 8.8: SP 1.3: P 7.5: RC 0.9.* Refers to G. Champe, C. B. Patterson.

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(1) KLOBERG, EDWARD, 11 West Forty-second St., New York City. (Age 47. Born New York City.) Prof. Engr., New York State.—1906 B. S. C. E. Cooper Union. *TT 4: P 4.*—Dec. 1903 to Sept. 1906 with Board of Water Supply, Brooklyn, as Instrumentman, in charge of field party laying steel main (8 miles) Rosedale to Amityville, N. Y., railroad crossings, reinforced concrete culverts, etc., drafting, computing probable distribution of water from Catskill Supply, etc. *TT 1.5: SP 1.1: P 0.4: D 0.4.*—Sept. 1906 to June 1908 student, Massachusetts Inst. of Technology; vacations with Board of Water Supply.—June 1908 to April 1913 with Water Supply of Manhattan, in charge of high-pressure water mains, 26 miles to new system, Designing Engr. on proposed Jerome Park filters. *TT 4.8: P 4.8: RC 3: D 1.1.*—April 1913 to Jan. 1914 Chf. Engr., Jas. C. Harding, Cons. Engr. for City of Schenectady, N. Y., supervising and directing construction of pumping-station and sewage-disposal plant, also constructing pumping-station and sewer system for Gen. Elec. Co. *TT 0.7: P 0.7: RC 0.7.*—Jan. 1914 to Aug. 1918 Draftsman, Foreman and Supt., Smith Hauser & MacIsaac, on construction of William St. Subway, Beekman St. to Hanover Sq., New York City. *TT 4.6: P 4.6: RC 2: D 0.6.*—Sept. 1918 to Jan. 1921 Constr. Engr. and Directing Engr., Beech-Nut Packing Co., Canajoharie, N. Y., pitometric studies on 12-mile wooden pipe line, sanitary condition of water shed and installation of Chicle Plant in Bush Terminal, Brooklyn. *TT 2.3: P 2.3: RC 2: D 0.3.*—Jan. 1921 to March 1925 Engr. and Supt. with Spencer, White & Prentis, Foundation & Underpinning Engrs., New York City. *TT 4.2: P 4.2: RC 2: D 0.2.*—March 1925 to Oct. 1928 Underpinning Engr. with Heyman & Goodman Co., underpinning buildings on 8th Ave. Subway and elevated R. R., 111th to 122d St., underpinning, maintenance and transfer of railroad to roof of new subway. *TT 3.6: P 3.6: RC 3: D 0.6.*—Oct. 1928 to March 1929 Supt.

and Underpinning Engr. with D. C. Serber Co., on 14th St. Subway, 6th to 8th Ave., New York City. *TT 0.4: P 0.4: RC 0.4: D 0.1.*—March 1929 to March 1930 Vice-Pres., International Eng. Corporation, New York City, underpinning for J. F. Coogan Co. and Necaro Co. on Schermerhorn St. Subway, Brooklyn. *TT 1: P 1: RC 1.*—March 1930 to date Contr. Engr. (Edward Kloberg, Inc.), New York City, underpinning buildings for Geo. H. Flinn Corporation, Slattery Daino Co., Kew Gardens, Golden Constr. Co., Locust St., Subway, Philadelphia, and all buildings on South Broad St. Subway, Philadelphia, Pa. *TT 1.8: P 1.8: RC 1: D 0.8.*—*TT 28.9: SP 1.1: P 27.8: RC 15.1: D 4.1.* Refers to C. L. Bogert, W. W. Brush, R. W. Greenlaw, J. C. Meem, R. Ridgway, J. F. Sanborn, J. W. Smith.

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(3) **LAIDLAW, DOUGLAS STAUNTON,** P. O. Box 50, Beauharnois, Que., Canada. (Age 26. Born Toronto, Ont., Canada.) 1928 B. A. Sc., Univ. of Toronto. *TT 4: P 4.*—April 1928 to July 1929 with Ontario Hydro-Elec. Power Comm., about 6 months as Laboratory Asst., Structural Testing Laboratories, on concrete tests, shop and field inspections of steel, then Draftsman, Hydr. Dept., on power-house substructures. *TT 0.6: SP 0.6.*—Sept. 1929 to Aug. 1930 with B. G. de Hueck & Co., Ltd. (De Hueck & Mattice), Cons. Engrs., Montreal, on structural and architectural drafting, some design, estimating, surveys, etc. *TT 0.5: SP 0.5.*—Aug. 1930 to Jan. 1931 with United Engrs. & Constructors (Canada), Ltd., Montreal, on structural and some architectural drafting. *TT 0.2: SP 0.2.*—Jan. to May 1931 (short periods) drafting, surveying, etc.—May 1931 to date Draftsman, Railway and Bridge Depts., Beauharnois (Que.) Constr. Co., designing and drafting on small bridges and culverts, for power development. *TT 0.4: SP 0.4.*—*TT 5.7: SP 1.7: P 4.* Refers to C. D. Babcock, O. B. Bestor, F. H. Cothran, T. H. Hogg, C. H. Mitchell, C. R. Young.

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(14) **LAWRENCE, HARVEY TICE,** 522 West Jefferson St., Mangum, Okla. (Age 30. Born Villisca, Iowa.) 1925 B. S. in Eng., and C. E., Univ. of Mo. *TT 4: P 4.*—June 1925 to May 1926 Jun. Highway Engr., Bridge Office, Illinois Highway Dept., Springfield, Ill., on design and drafting. *TT 0.5: SP 0.5.*—May 1926 to Jan. 1929 Chf. Engr., Edward W. Gantt Co., Cons. Engrs., Oklahoma City, Okla., on designs, estimates and in charge of field work. *TT 2.7: P 2.7: RC 2.7: D 1.*—Jan. to May 1929 private consultation work, design and construction of water-softening plant, gravity zeolite system. *TT 0.3: P 0.3: RC 0.3.*—May 1929 to date City Engr. and City Mgr., Mangum, Okla., in charge of all design, construction of municipal water, electric and gas systems, etc. *TT 2.7: P 2.7: RC 2.7.*—*TT 10.2: SP 0.5: P 9.7: RC 5.7: D 1.* Refers to G. F. Burch, L. M. Bush, A. L. Hyde, A. R. Losh, E. J. McCaustland, H. K. Rubey.

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(15) **LOTT, JAMES GUY,** Marfa, Tex. (Age 40. Born Leesville, Tex.) 1909 to 1913 student in C. E., Agri. & Mech. Coll. of Texas. *TT 1.5: P 1.5.*—July 1913 to April 1914 Rodman, San Antonio, Uvalde & Gulf R. R. *TT 0.4: SP 0.4.*—June 1914 to Aug. 1915 Instrumentman, City of Beeville, surveys and sewer system, staking and computations. *TT 0.6: SP 0.6.*—Sept. 1915 to Jan. 1916 Draftsman, Goliad County Highways, general office work. *TT 0.1: SP 0.1.*—Jan. to Sept. 1916 Chainman, Rodman and Instrumentman, Interstate Commerce Comm., Kansas City, Mo. railroad valuation. *TT 0.3: SP 0.3.*—Sept. 1916 to May 1917, Asst. Pilot, Atchison, Topeka & Santa Fe R. R., Amarillo, Tex., checking Government Valuation Party in field. *TT 0.3: SP 0.3.*—May 1917 to July 1919 1st Lieut. and Capt., U. S. Army, on construction work in France. *TT 2.2: P 2.2: RC 2.2.*—July 1919 to Feb. 1922 with Texas Highway Dept., until Nov. 1920 as Res. Engr., on location, plans, specifications, estimates and construction, then Div. Engr., Amarillo, Tex., general charge of construction and maintenance of highways in Div. No. 4. *TT 2.5: P 2.5: RC 2.5: D 1.3*—Feb. 1922 to Aug. 1923 Engr.-in-Chg. on highway construction in Johnson County, Tex. *TT 1.6: P 1.6: RC 1.6.*—Jan. 1924-March 1930 County Engr., Dummit County, Tex., on location, plans, specifications, estimates and construction on State highways in county. *TT 6.3: P 6.3: RC 6.3: D 6.3.*—April 1930 to date Res. Engr., Texas Highway Dept., on location, plans, specifications, estimates and construction in Dimmit, Zavala, Uvalde, Brewster, Presidio and Jeff Davis Counties. *TT 1.8: P 1.8: RC 1.8: D 1.8.*—*TT 17.6: SP 1.7: P 15.9: RC 14.4: D 9.4.* Refers to G. G. Edwards, G. Gilchrist, M. B. Hodges, T. E. Huffman, G. M. Jowers, T. J. Kelly, O. A. Seward, Jr.

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(3) **MACDONALD, JOHN,** 28 Union Ave., Schenectady, N. Y. (Age 53. Born Glasgow, Scotland.) 1906 B. S. in Civ. Eng., N. Y. Univ. *TT 4: P 4.*—Sept. 1895 to Sept. 1902 Apprentice, Mechanic and Asst. Foreman with C. T. Wills, Inc., New York City. *TT 3.5:*

SP 3.5.—June 1906 to Sept. 1924 Constr. Supt., successively with Bliss-Griffiths Co., on a department store in New York, a factory in Hanover, Pa., and a high school in Rutland, Vt. (2 years), with Deisler & Stephenson on a residence for H. Bloomingdale (1 year) with C. T. Willis, Inc., on a studio for Mrs. Harry P. Whitney, Roslyn, N. Y., Professional Building, and residences for James Speyer and Adolph Lewisohn, New York City (6 2/5 years) and with Thompson Starrett Co., on residences for Chas. McNeil and Otto Kahn in New York City, a railroad station in New Haven, Conn., the Internation G. E. Co. Bldg. in Schenectady, N. Y., Sterling Laboratory, Sterling Hall of Medicine and a power house for Yale Univ. (8 2/5 years). *TT. 17.8: P 17.8: RC 17.8.*—Sept. 1924 to date Thompson Starrett Associate Prof. of Civ. Eng. at Union Coll., until 1927 being Asst. Prof. and since then Associate Prof., in charge of course in building construction (since 1924) and of structural design (since 1927). *TT 7.3: P 7.3: RC 3: D 5.*—*TT 32.6: SP 3.5: P 29.1: RC 20.8: D 5.* Refers to J. L. Bogart, R. A. Hall, A. de H. Hoadley, H. Miller, H. A. Schauffler, W. C. Taylor, A. Vogel.

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(5) MACKEY, LINCOLN, 2512 Que St. N. W., Washington, D. C. (Age 30. Born Rosslyn, Va.) 3 years Special Student, George Washington Univ.—June to Nov. 1917, April to Dec. 1918 and April to Sept. 1919 Chainman, U. S. Land Office, Indian Reservation Surveys. *TT 0.8 SP 0.8.*—Sept. to Nov. 1919 Levelman, Redwood County, Minn., and April 1920 to Jan. 1921 Deputy County Surveyor on road location, Cass County, Minn. *TT 0.5: SP 0.5.*—June to Oct. 1921 and May to Oct. 1922 with Iowa State Highway Comm., in 1921 as Instrumentman and Inspector on construction, and in 1922 Chf. of Party on surveys. *TT 0.6: SP 0.1: P 0.5.*—Oct. 1922 to June 1923 Chf. of Party and Project Engr., and May 1925 to Nov. 1926 Chf. of Party, Virginia Highway Comm., on location, design and construction, including design of small drainage structures. *TT 2: P 2: RC 2: D 2.*—June 1923 to April 1925 Civ. Engr. and Surveyor in Arlington County, Va., on highway location, design and construction, sewer surveys, design and construction. *TT 1.8: P 1.8: RC 1.8: D 1.8.*—Nov. 1926 to April 1927 Engr.-in-Chg., San Francisco Mines of Mexico, Ltd., surface surveys, precise leveling, design and construction of roads and sewers. *TT 0.5: P 0.5: RC 0.5: D 0.5.*—April to Oct. 1927 and March 1928 to Dec. 1929 Asst. County Engr., Fairfax County, Va., in charge of surveys, bridge designs and highway construction. *TT 2.3: P 2.3: RC 2.3: D 2.3.*—Oct. 1927 to Feb. 1928 Jun. Civ. Engr., Navy Dept., on highway location, design and construction, including design of necessary drainage structures, also location of magazine sites for high explosives. *TT 0.3: P 0.3: RC 0.3: D 0.3.*—Dec. 1929 to July 1930 Chf. of Party, Virginia Highway Comm., on location, design and construction. *TT 0.7: P 0.7: RC 0.7: D 0.7.*—Aug. 1930 to date County Engr., Arlington County, Va., in charge of roads, sewers, water and of design and construction of extensions. *TT 1.3: P 1.3: RC 1.3: D 1.3.*—*TT 10.8: SP 1.4: P 9.4: RC 8.9: D 8.9.* Refers to E. C. Dunn, T. Ellett, J. M. Harbert, B. P. Harrison, W. H. Richards, Jr., H. G. Shirley.

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(1) McMAHON, HARRY RAYMOND, 49 Summit Road, Allwood, Clifton, N. J. (Age 24. Born Harlan, Iowa.) 1931 B. S., Iowa State Coll. *TT 4: P 4.*—Aug.–Dec. 1928 and June to Oct. 1929 with Eng. Experiment Station, Iowa State Coll., Ames, Iowa, until Dec. 1928 as Asst. to Field Engr., on instrument work, etc., securing, compiling and interpreting field data on experimental work, then Asst. to Bulletin Editor. *TT 0.2: SP 0.2.*—June 1930 to Jan. 1931 Draftsman, Michigan State Highway Dept., Lansing, Mich., computing earthwork and construction materials and drafting for road plans. *TT 0.6: P 0.6: RC 0.6.*—*TT 4.8: SP 0.2: P 4.6: RC 0.6.* Refers to T. R. Agg, R. A. Caughey, J. S. Dodds, W. L. Foster, A. H. Fuller.

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(1) MATSUI, YASUO, 350 Fifth Ave., New York City. (Age 54. Born Shima, Japan.) 1898 graduated from Keiwo Univ. Student Univ. of California (2½ years) and Massachusetts Inst. Technology (1 year). *TT 1.5: P 1.5.*—1904 to 1911 Archt. Designer with McKim, Mead & White, George B. Post, Ernest Flagg and Warren & Wetmore, all in New York City. *TT 6: P 6: RC 4: D 6.*—1911 to 1912 in partnership with Wengenroth & Matsui, Archt. and Engr. *TT 1: P 1: RC 1: D 1.*—1912 to 1917 with Starrett & Van Vleck, Archts., New York City, in charge of office and on design. *TT 5: P 5: RC 5: D 5.*—1917 to 1920 Cons. Archt. and (after 1918) also Executive Adviser to George A. Fuller Co., Bldrs. *TT 3: P 3: RC 3: D 1.*—1921 to 1925 Managing Director, and 1925 to date President, F. H. Dewey & Co., Archts. and Engrs., New York City; as Archt. and Engr. designed numerous important buildings, including office, factory, bank and automobile service buildings, a general hospital, a church, apartment houses, residences, underpinning of

U. S. Sub-Treasury Bldg., etc. and as Associate Archt. designed and erected an office building, Brown Bros. Bldg., Starrett-Lehigh Bldg. and Bank of Manhattan Bldg., all in New York City. *TT 12: P 12: RC 12: D 12.*—*TT 28.5: P 28.5: RC 25: D 25.* Refers to A. J. Post, N. A. Richards, C. B. Spencer, H. V. Spurr, W. A. Starrett, S. C. Weiskopf, L. White.

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(11) **MAUZY, HARRIS KENNETH,** 1306 Pine St., South Pasadena, Cal. (Age 26. Born Chicago, Ill.) 1930 B. S. in Eng., Cal. Inst. Tech. *TT 4: P 4.*—Aug. to Sept. 1930 Chainman, Rodman and Stadia Party Recorder with Franklin Thomas.—Sept. 1930 to date Draftsman, California Highway Comm., two months on cross sections and computations, then with Right of Way Dept., mapping, etc. *TT 0.7: SP 0.7.*—*TT 4.7: SP 0.7: P 4.* Refers to S. V. Cortelyou, A. D. Griffin, R. R. Martel, W. W. Michael, F. Thomas.

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(9) **MOORE, RAYMOND LeROY,** 2005 Cherry St., Toledo, Ohio. (Age 28. Born Columbus, Ohio.) Oct. 1923 to June 1924 student, Ohio State Univ. *TT 0.5: P 0.5.*—Aug. 1922 to Oct. 1923 and Aug. 1924 to Sept. 1925 with Pennsylvania R. R., first as Electrician's Helper, and after Aug. 1924 on general electric work, including plant and equipment maintenance. *TT 1.5: SP 0.7: P 0.8.*—Oct. 1925 to Jan. 1926 Inspector, H. G. Fugate Co., West Palm Beach, Fla., on sub-division work, including surveys, street construction, layout, borings for bridge site investigation, etc. *TT 0.2: P 0.2.*—Jan. to April 1926 with Realty Development Corporation, West Palm Beach, Fla., as Res. Party Chf. on Heota Development at Taft, Fla. *TT 0.3: P 0.3: RC 0.3.*—Oct. 1928 to May 1929 Field Engr., City Eng. Dept., Pontiac, Mich., on preliminary surveys and investigations. *TT 0.6: P 0.6: RC 0.6.*—April 1926 to Oct. 1928 and May 1929 to date with R. H. Randall & Co., Toledo, Ohio, first on geodetic and topographic surveys in Columbus, Ohio, Evansville, Ind. and Pontiac, Mich., and since May 1929 Asst. Res. Engr., Akron, Ohio, Topographic Engr. and (part of time) Asst. Engr. in charge, Willacy County, Tex., Res. Engr., Binghamton, N. Y., Asst. Res. Engr., Lucas County, Ohio, on topographic and triangulation surveys; at present Res. Engr. in Toledo, on precise traverse survey. *TT 5.3: P 5.3: RC 4.3: D 1.*—*TT 8.4: SP 0.7: P 7.7: RC 5.2: D 1.* Refers to R. W. Abbett, R. C. Chaney, W. S. Dix, E. H. Prentice, R. H. Randall, R. C. Sweeney, G. D. Whitmore.

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(8) **MOSIER, RAY ROSEVELT,** 901 S. Central St., Paris, Ill. (Age 27. Born Cherokee, Iowa.) 1930 B. S. in C. E., Univ. of Wyo. *TT 4: P 4.*—Sept. to Dec. 1924 Rodman, Sigmund Niser Constr. Co., Denver, Wyo. *TT 0.1: SP 0.1.*—July 1926 to March 1927 Transmission Clerk, Midwest (Wyo.) Refining Co., on map work, etc. *TT 0.2: SP 0.2.*—June to Nov. 1929 Bridge and Building Timekeeper, Union Pacific R. R., on construction of new viaduct. Aug. 1930 to date with Illinois Div. of Highways, until July 1931 Jun. Engr., being Mixer Inspector and Proportioning Engr. on reinforced concrete pavement construction and bridge inspection, also Draftsman, then Res. Engr., in charge of reinforced concrete bridge and miscellaneous structures, reports and estimates, and since Sept. 1931 Maintenance Engr. (Jun.), on surveys, drafting and in office. *TT 1.5: P 1.5: RC 0.2.*—*TT 5.8: SP 0.4: P 5.4: RC 0.2.* Refers to C. H. Apple, J. C. Fitterer, R. D. Goodrich, H. T. Person.

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(1) **MOTA, CANDELARIO CALOR,** Box 131, Marina Station, Mayaguez, Porto Rico. (Age 33. Born Aguadilla, Porto Rico.) 1924 B. S., and 1927 M. S., Mass. Inst. Tech. *TT 4: P 4.*—July 1924 to Sept. 1925 Asst. Engr., Dept. of Interior, San Juan, Porto Rico, on design of sewerage and water-works, etc. *TT 1.3: P 1.3: D 1.3.*—Oct. 1925 to June 1926 Instructor, Sept. 1926 to May 1931 Asst. Prof., and Sept. 1931 to date Associate Prof., in Civ. Eng., Univ. of Porto Rico, in charge of classes in bridge and structural design, reinforced concrete, theory of structures, and graphic statics; summer 1929 with Robinson & Steinman, New York City, on design of bridges. *TT 4.9: P 4.9: RC 4.7: DO 2.*—*TT 10.2: P 10.2: RC 4.7: D 1.5.* Refers to R. R. Casellas, R. A. Gonzalez, E. Ortega-Rosado, C. M. Spofford, E. Totti y Torres, J. A. L. Waddell.

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(1) **NELSON, ARTHUR MANFRED,** 14 McIntyre St., Bronxville, N. Y. (Age 29. Born Boston, Mass.) 1924 B. S. in C. E., Univ. of Mich. *TT 4: P 4.*—July 1924 to Oct. 1928 Designer, Bridge Div., Michigan State Highway Dept., on design and detail plans and estimates for reinforced concrete and structural steel bridges and grade separation structures. *TT 4.3: P 4.3: RC 1.*—Oct. 1928 to April 1929 Designer and Detailer, Boston & Maine R. R., on design, plans and estimates for timber, reinforced concrete and structural

steel bridges, turntables, trestles. *TT 0.5: P 0.5.*—April 1929 to date Designer and Detailer, New York Central R. R. Co., on design and plans for bridges, grade separation structures, etc. *TT 2.8: P 2.8.*—*TT 11.6: P 11.6: RC 1.* Refers to C. A. Melick, H. E. Riggs, Z. H. Sikes, W. L. Unger, H. T. Welty.

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(1) O'LEARY, WILLIAM ALOYSIUS, 788 Riverside Drive, New York City. (Age 30. Born Elizabeth, N. J.) 1922 B. S. in Civ. Eng., Villanova, Coll. *TT 4: P 4.*—July-Aug. 1922 Jun. Engr., New Jersey State Highway Comm., Lakewood, N. J., laying out concrete roads and inspecting construction and materials. *TT 0.1: P 0.1.*—Sept. 1922 to June 1923 Foreman and Gen. Foreman, Public Service Production Co., Newark, N. J., supervising construction of concrete highways, sewers and culverts at Toms River, Barnegat and Tuckerton, N. J. *TT 0.8: P 0.8: RC 0.8.*—Aug. 1923 to Aug. 1926 Structural Draftsman, Transit Comm., State of New York, and Board of Transportation, New York City, and Aug. 1926 to date Structural Designer, Board of Transportation, checking and preparing studies, contract and construction drawings for new sewers, subway drainage, subsurface relocation and gas-bypassing in connection with Third Independent Subway System in New York City; Oct. 1924 to Aug. 1926 supervising work; since Jan. 1930 making studies in connection with drainage and water supply, new sewers and gas-bypassing in connection with proposed Midtown and Narrows Vehicular Tunnels; past few months also studying 62 years of rainfall records in Borough of Manhattan, to determine rainfall intensity formulas for Rational Formula for runoff. *TT 8.1: SP 0.3: P 7.8: RC 7.3: D 7.8.*—*TT 13: SP 0.3: P 12.7: RC 8.1: D 7.8.* Refers to A. H. Bull, C. E. Conover, J. H. Quimby, A. I. Raisman, T. F. Weiss.

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(11) PINYAN, RONALD AUGUST, Beaumont, Cal. (Age 24. Born Los Angeles, Cal.) 1931 B. S. in C. E., Univ. of So. Cal. *TT 4: P 4.*—Feb. 1926 to Sept. 1927 Jun. Engr., Scofield Eng. Constr. Co., Los Angeles, Cal., on building construction. *TT 0.8: SP 0.8.*—Oct. 1931 to date Jun. Engr., Metropolitan Water Dist. of Southern California, Beaumont, calculating, drafting and estimating in connection with aqueduct from Calorado River to Los Angeles. *TT 0.2: SP 0.2.*—*TT 5: SP 1: P 4.* Refers to B. A. Eddy, R. M. Fox, D. M. Wilson.

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(11) SCHUMACHER, KARL FRITZ, 197 East Ninth St., San Bernardino, Cal. (Age 28. Born Mess Kirch, Germany.) 1929 B. S., Cal. Inst. Tech. *TT 4: P 4.*—Jan. 1925 to Feb. 1927 Chainman and Instrumentman on subdivision and construction, until June 1925 with Mission Beach Co., then with John P. Mills. *TT 1: SP 1.*—Feb. to March 1927 Hydrographer, and Aug. to Sept. 1927 Computer, Vail Co. *TT 0.1: SP 0.1.*—March to July 1927 Instrumentman for Thomas H. King, San Diego, Cal. *TT 0.2: SP 0.2.*—Sept. 1927 to Jan. 1928 (part of time) Chainman with O. A. Loebenstein, San Diego. *TT 0.1: SP 0.1.*—Jan. to Sept. 1928 Chf. of Transit Party for Governor of Lower California, Mexico, on highway location. *TT 0.3: SP 0.3*—June to July 1929 Chainman, Atchison Topeka & Santa Fe Ry. *TT 0.1: SP 0.1.*—Aug. 1929 to date Jun. Engr., Water Resource Branch, U. S. Geological Survey, stream gauging. *TT 2.5: P 2.5.*—*TT 8.3: SP 1.8: P 6.5.* Refers to H. F. Hill, Jr., J. C. Hoyt, H. D. McGlashan, F. Thomas, H. C. Troxell.

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(9) SCHMUCKER, LEROY LELAND, 26 East Tallmadge Ave., Akron, Ohio. (Age 31. Born Stryker, Ohio.) 1927 C. E., Univ. of Akron. *TT 4: P 4.*—Nov. 1923 to July 1928 (until June 1927 while student) Asst. Engr., Baltimore & Ohio R. R. on instrument work, drafting and yard and industrial-track layouts. *TT 1.1: P 1.1: RC 0.3.*—Oct. 1928 to Aug. 1931 Asst. Estimator, Carmichael Constr. Co., estimating, detailing, expediting, and designing. *TT 2.8: P 2.8: RC 1.1: D 0.3.*—Sept. 1931 to date Asst. Mgr., First-Central Trust Co., on construction. *TT 0.3: P 0.3: RC 0.3.*—*TT 8.1: P 8.1: RC 1.7: D 0.3.* Refers to F. E. Ayer, J W Bulger, R C. Durst, M. P. Lauer, R. E. Wilson.

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(1) SFILIGOJ, BOGOMIR, 32 Overpeck Ave., Ridgefield Park, N. J. (Age 35. Born Medana, Italy.) 1924 C. E., Czech Pol. Inst. of Brno, Czechoslovakia. *TT 4: P 4.*—Oct. 1914 to April 1915 student, Pol. Inst. of Graz in Austria.—May 1924 to April 1929 in private practice as Civ. Engr. and Surveyor, Gorizia, Italy, until Sept. 1926 in partnership with Domenico Rocco, surveying, estimates and plans for small bridges and buildings. *TT 5: P 5: RC 5: D 5.*—June 1929 to May 1930 Draftsman with Gibbs & Hill, Cons. Engrs., on steel structures and layouts in connection with railroad electrification. *TT 0.5: SP 0.5.*—

May 1930 to date Designer with George F. Hardy, Cons. Engr., on water power plants. *TT 1.7: P 1.7: D 1.7.*—*TT 11.2: SP 0.5: P 10.7: RC 5: D 6.7.* Refers to R. Bolaffio, A. G. Ready, A. H. Reeves, K. W. Ross, T. T. Whittier.

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(13) SMIRNOFF, VALENTINE FEDOR, 306 Fourth Ave., San Francisco, Cal. (Age 26. Born Mitava, Russia.) 1930 B. S., Univ. of Cal. *TT 4: P 4.*—Sept. 1931 to Jan. 1932 Chainman, San Francisco Water Dept., on construction of Crystal Springs Aqueduct. *TT 0.2: SP 0.2.*—*TT 4.2: SP 0.2: P 4.* Refers to C. Derleth, Jr., B. A. Etcheverry, S. T. Harding, B. Jameyson, P. A. Swafford.

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(4) SMITH, DAVID OLIVER, 410 North Matlack St., West Chester, Pa. (Age 34. Born Mechanicsburg, Pa.) 1926 received Diploma in Structural Eng., Drexel Inst. Evening School.—April 1920 to March 1921, May to Dec. 1921 and April to Dec. 1922 on Pennsylvania State Highway construction, until March 1921 with Langthorne Constr. Co., New York City, 5 months as Timekeeper, then Chf. Clerk, in charge of field office, May to Dec. 1921 with Merdinger Constr. Co., Bethlehem, Pa., 4 months as Timekeeper, then Div. Supt., in charge of field force, and after April 1922 with Development & Constr. Co., Baltimore, Md., 3 months as Timekeeper, then Field Engr., furnishing lines and grades and preparing monthly work estimate. *TT 1.7: SP 0.4: P 1.3: RC 1.3.*—Jan. 1923 to July 1927 with Turner Constr. Co., Philadelphia, Pa., until Dec. 1923 as Timekeeper, checking materials, furnishing lines and grades, checking sub-contractors' work on reinforced concrete building, etc., Dec. 1923 to June 1925 Cost Engr., preparing cost accounts and analyses for completed cost of all types of buildings, June 1925 to June 1926 furnishing lines and grades, checking work on building construction, and after June 1926 Asst. Supt., checking plans, co-ordinating, supervising and checking work of sub-contractors on building construction, etc. *TT 4: SP 0.4: P 3.6: RC 3.*—July 1927 to March 1928 Supt. of Constr., with Joseph W. Pomraning, Harrisburg, Pa., on erection of high-school building. *TT 0.7: P 0.7: RC 0.7.*—March to June 1928 in private practice as Contr., estimating and bidding on building projects. *TT 0.3: P 0.3: RC 0.3.*—June 1928 to Sept. 1929 Res. Engr., supervising construction of two school buildings for Board of School Directors, Manheim Township, Pa. *TT 1.3: P 1.3: RC 1.3.*—Sept. 1929 to July 1930 Supt. of Constr., Bureau of Eng. & Constr., Dept. of Property and Supplies, Commonwealth of Pennsylvania, Harrisburg, Pa., on erection of buildings, power plant and sewage-disposal plant. *TT 0.8: P 0.8: RC 0.8.*—July 1930 to date Div. Engr., Bituminous Service Co., West Chester, Pa., on estimating, designing and supervising construction of macadam roads. *TT 1.5: P 1.5: RC 1.5: D 1.*—*TT 10.4: SP 0.9: P 9.5: RC 8.9: D 1.* Refers to E. R. Bear, B. A. Owen, R. Radbill, H. C. Seward, A. J. Warlow.

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(7) THOMSON, GORDON HOPE, Univ. of Minnesota, Minneapolis, Minn. (Age 25. Born Ballinger, Tex.) 1930 B. S. in C. E., Tex. Tech. Coll. *TT 4: P 4.*—Sept. 1930 to June 1931 and Sept. 1931 to date Research Fellow, until June 1931 in Highway Eng., then in Structural Eng., Univ. of Minnesota. *TT 1.1: P 1.1.*—*TT 5.1: P 5.1.* Refers to F. Bass, J. I. Parcel.

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(1) WALLIN, FRANK ALTON, Box 266, Boonton, N. J., (Age 27. Born Anderson, Ind.) 1931 B. S. C. E., Purdue Univ. *TT 4: P 4.*—April to Oct. 1924 Chainman, Rodman and Levelman on topographical survey of Flint River for proposed reservoir at Genesee, Mich., G. A. Johnson, Cons. Engr. *TT 0.2: SP 0.2.*—April 1925 to Sept. 1926 on construction of sewage-disposal plant, Trenton, N. J., being Timekeeper, Cost Clerk and Asst. to Contr.'s Engr., J. P. White Co., Passaic, N. J., Contrs., G. A. Johnson, Cons. Engr. *TT 0.8: SP 0.8.*—July to Nov. 1931 Asst. Engr. on construction of interceptor sewers and sewage-disposal plant, Norwalk, Conn. *TT 0.3: P 0.3.*—Nov. 1931 to date Field Engr. on topographical survey and field study for proposed reservoir at Split Rock, N. J., G. A. Johnson, Cons. Engr. *TT 0.3: P 0.3.*—*TT 5.6: SP 1: P 4.6.* Refers to F. S. Childs, R. M. Genthon, G. A. Johnson.

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(15) WAMSLEY, DONALD CARPENTER, 5018 Airline Rd., Dallas, Tex. (Age 31. Born Mattoon, Ill.) 1925 B. S., Mass. Inst. Tech. *TT 4: P 4.*—Oct. 1922 to Sept. 1924 and July 1925-Nov. 1931 with Missouri Pacific R. R. Co., first as Rodman, and after July 1925 Instrumentman and Asst. Engr., Constr. Dept., on location and construction. *TT 7.2: SP 1: P 6.2: RC 2.7.*—*TT 11.2: SP 1: P 10.2: RC 2.7.* Refers to C. B. Breed, W. J. Burton, K. L. DeBlois, E. A. Hadley, C. S. Sample, H. A. Sargent, R. C. White, S. L. Wonson.

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(1) **WEINSTOCK, ISIDOR LAWRENCE,** 1402 Ave. K, Brooklyn, N. Y. (Age 21. Born New York City.) 1931 B. S., and 1932 C. E., Coll. of City of N. Y. *TT 4: P 4.*—Sept. 1930-June 1931 (while student) Asst. Field Instructor of surveying, Coll. of City of New York.—Summers 1929 and 1931 Jun. Asst. Civ. Engr., Grade 1 and 2 (field), Div. of Highways, New York State Dept. of Public Works, being Inspector of road construction, Transitman, Levelman, Note-keeper, Rodman, etc.—*TT 4: P 4.* Refers to R. E. Goodwin, F. O. X. McLoughlin.

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(9) **YANDA, ALFRED DANIEL,** 4293 East One Hundred Twenty-eighth St., Cleveland, Ohio (Age 24. Born Cleveland, Ohio.) Sept. 1924 to Nov. 1925 student, Ohio State Univ. *TT 0.5: P 0.5.*—Jan. to Feb. 1926 Draftsman with Robb & Buchanan, Civ. Engrs., on graphs and subdivision plats. *TT 0.1: SP 0.1.*—Feb. 1926 to April 1927 with Carter and Damerow, Inc., as Draftsman, Rodman and (about 6 months) Chf. of Party, on subdivisions, streets, bridges, cross-state highway, mosquito control project, river soundings, etc. *TT 0.5: SP 0.5.*—May 1927 to Dec. 1928 Senior Draftsman, and Dec. 1928 to date Senior Asst. Civ. Engr., Cuyahoga County, Cleveland, Ohio, drafting, earthwork, plans, profiles, plats and maps of county and state road work; Engr. on design of pavements and construction plans, including design of storm sewers and work incidental to paving, sketching design of pavement, designing or checking construction plans, storm sewer for road and estimating materials and cost for road job. *TT 4: SP 0.8: P 3.2: RD 3.2: D 3.2.*—*TT 5.1: SP 1.4: P 3.7: RC 3.2: D 3.2.* Refers to E. C. Blosser, R. C. Chaney, G. W. Hamlin, H. G. Reitz, W. J. Watson.

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(1) **ZISMAN, JOSHUA FELIX,** 3120 Buhre Ave., New York City. (Age 36. Born Jassy, Roumania.) 1920 received diploma, Montreal Technical School.—Dec. 1916 to Nov. 1918 Draftsman, and Feb. 1923 to Nov. 1927 Estimator and Designer on steel work, for American Tool Co., Pawtucket, R. I. *TT 5.8: SP 1: P 4.8: RC 2: D 2.8.*—June 1920 to Nov. 1922 Draftsman and Designer, P. Herdman Co., Montreal, Que., on building construction. *TT 2.2: SP 0.2: P 2: RC 1: D 2.*—May to Dec. 1928 Steel Detailer, Truscon Steel Co., Montreal, on reinforcing steel for concrete structures. *TT 0.3: SP 0.3.*—April to Dec. 1929 Structural Steel Draftsman, New York Edison Co., New York City, on structural steel drawings for power plants. *TT 0.4: SP 0.4.*—April to Dec. 1930 Checker and Estimator, White Constr. Co., New York City, checking drawings and estimating building materials. *TT 0.8: P 0.8.*—May 1931 to date Inspector, Dock Dept., City of New York, inspecting construction of docks and piers. *TT 0.3: SP 0.3.*—*TT 9.8: SP 2.2: P 7.6: RC 3: D 4.8.* Refers to R. L. Bertin, E. J. Critzas, W. T. Doran, C. B. Galvin, C. W. Haasis.

FOR TRANSFER

FROM THE GRADE OF ASSOCIATE MEMBER

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(14) **BARDSLEY, CLARENCE EDWARD S.,** Assoc. M., 404 Walnut St., Rolla, Mo. (Elected Aug. 26, 1929.) (Age 37. Born St. Louis, Mo.) 1920 B. S. in C. E., 1922 C. E., and 1924 M S., Mo. School of Mines and Metallurgy. 1926 Sc. D. in Civ. Eng., Univ. of Mich. *TT 4: P 4.*—Summers 1927 and 1928 student, Northwestern Univ. 1913 to 1914 and summers 1914 and 1915 Rodman, successively with Missouri Pacific R. R. and Wabash R. R. *TT 0.6: SP 0.6*—Summer 1917 Draftsman, Pennsylvania R. R., 1917 Field Asst., 1918 Jun. Topographer, and 1924 Associate Topographer, U. S. Geological Survey, mapping, etc. *TT 2.5: P 2.5: RC 0.5.*—1919 to 1920 Instructor in mathematics and civil engineering, 1920 to 1924 Asst. Prof. of Topographic Eng., 1925 to 1928 and 1929 to 1930 Asst. Prof., and 1930 to 1931 Associate Prof. of Civ. Eng., and Sept. 1931 to date Prof. of Hydr. Eng., Missouri School of Mines and Metallurgy; also 1925 to 1926 National Slag Association Fellow, and summer 1926 Associate Prof. of Highway Eng and Transportation, Univ. of Michigan; 1926 to 1928 on property and drainage surveys; 1927 to 1928 City Engr., Rolla, Mo., on design of curb and gutter, street paving, culverts, sewers, water lines, public utilities, surveys, etc.; 1928 Acting County Highway Engr., Phelps County, Mo. and Expert Witness in Court on earthwork dispute and railroad property case; 1928 to 1929 John R. Freeman Scholarship student for study of European hydraulics in Germany; June to Aug. 1930 and May to Aug. 1931 Associate Hydr. Engr., Corps of Engrs., U. S. War Dept., Kansas City, Mo., studying canalization of lower Missouri River and at U. S. Waterways Experiment Station, Vicksburg, Miss., and (after May 1931) studying feasibility of constructing pilot channels in the Boeuf and Atchafalaya Diversion

Basins and soil-erosion studies, and Technical Advisor to Laboratory; since Sept. 1929 Deputy County Engr., Phelps County, Mo., also consulting practice, surveys, court cases, design and hydraulic practice. *TT 11.5: P 11.5: RC 11.5: D 1.2.*—*TT 18.5: SP 0.6: P 17.9: RC 12.2: D 1.2.* Refers to H. C. Beckman, H. E. Bilger, C. H. Birdseye, A. H. Blanchard, W. J. Burton, J. B. Butler, J. C. L. Fish, J. R. Freeman, E. G. Harris, R. L. Morrison, P. S. Reinecke, H. E. Riggs, J. G. Staack.

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(14) BUSH, LEE MARSHALL, Assoc. M., 1625 West Thirty-first St., Oklahoma City, Okla. (Elected Nov. 27, 1917.) (Age 43. Born Auburn, Kans.) 1911 B. S. in C. E., Univ. of Kans. *TT 4: P 4.*—Summer 1910 Levelman, Gulf & Northwestern Ry., also prepared right-of-way maps, profiles and construction estimates.—Feb. 1911 to April 1912 Rodman and Instrumentman, Kansas City Southern Ry. Co., on location, bank widening and track maintenance. *TT 1.2: P 1.2: RC 0.6: D 0.2.*—April to Nov. 1912 Asst. Engr., State of Kansas, on construction of State Fish Hatchery at Pratt, Kans., earth dam, retaining walls, pipe lines, concrete structures, etc. *TT 0.5: P 0.5.*—Nov. 1912 to Jan. 1914 Instrumentman, Rock Island Lines, on maintenance-of-way and structures, ballasting, bridge construction, flood-protection reports and investigations. *TT 1.2: P 1.2: RC 0.3.*—Jan. 1914 to April 1918 Jun. Engr., Interstate Commerce Comm., on railway valuation in western district, principally roadway and track inventory, some cost analysis. *TT 4.3: P 4.3: RC 3.*—April to June 1918 Asst. Engr., Missouri, Kansas & Texas Ry. Co., Waco, Tex., in charge of bridge rebuilding and culvert work. *TT 0.2: P 0.2: RC 0.2.*—June to Dec., 1918 2d Lieut., Field Artillery, U. S. Army.—Jan. to June 1919 Asst. Engr., Union Pacific Ry., Omaha, Nebr., on valuation, being Representative with Federal Govt. Valuation Engrs. *TT 0.4: P 0.4.*—July 1919 to Jan. 1921 Constr. Engr., Stamey Mackey Constr. Co., Hutchinson, Kans., in charge of Federal Aid highway construction. *TT 1.2: P 1.2: RC 1.2.*—March 1921 to May 1931 member of firm, V. V. Long & Co., Cons. Engrs., Oklahoma City, Okla., in charge of water-works, sewer, sewage disposal, electric light plants, gas systems and street paving for about 75 cities in six states. *TT 10.2: P 10.2: RC 10.2: D 10.2.*—May 1931 to date City Engr., Oklahoma City, in charge of street paving, sanitary and storm sewer construction, subways, bridges and construction of airport. *TT 0.7: P 0.7: RC 0.7: D 0.7.*—*TT 23.9: P 23.9: RC 16.2: D 11.1.* Refers to J. F. Brookes, P. K. Bunn, W. C. Burnham, V. H. Cochrane, C. A. Haskins, A. P. Learned, B. S. Myers, W. E. Price, E. R. Stapley, N. E. Wolfard.

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(15) CATE CHARLES EDWARD, Assoc. M., 1120 Avenida La Paz, Guadalajara, Jalisco, Mex. (Elected July 2, 1913.) (Age 47. Born New Orleans, La.) 1906 B. E. in C. E., Tulane Univ. *TT 4: P 4.*—June to Dec. 1906 Rodman, Chainman and Draftsman on railroad work. *TT 0.2: SP 0.2.*—Dec. 1906 to Jan. 1912 with Southern Pacific of Mexico, as follows: Dec. 1906 to May 1907 Topographer; May 1907 to Dec. 1908 and Dec. 1910 to Dec. 1911 Transitman, in charge of construction of a temporary bridge (twelve 70-ft. spans, deck plate girder) (5 months), on construction of Del Rio Nogales Branch, etc.; Dec. 1908 to March 1910 and Dec. 1911 to Jan. 1912 Asst. Engr., Sonora Div., on track renewal (260 miles), relocation and construction of a 2-mile loop, routine maintenance, etc. and (after Dec. 1911) relocated and constructed about 3 km. of line; April to Aug. 1910 Asst. Supt., in charge of maintenance of way, Sonora Div. *TT 4.6: P 4.6: RC 1.1: D 1.1.*—Sept. 1912 to Jan. 1913 Transitman on location, Christina Switchback and Florence-Superior Extension, and (Jan. 1913) Asst. Engr. in charge of construction of Miami Extension, Arizona Eastern R. R. *TT 0.3: P 0.3.*—Jan. 1913 to Jan. 1918 with S. P. of Mex., until Aug. 1916 as Asst. Supt., in charge of maintenance of way, structure and water service on Sonora Div., then Engr., M. of W., in direct charge of all maintenance work on system. *TT 5.1: P 5.1: RC 5.1: D 1.5.*—Jan. 1918 to Aug. 1919 with U. S. Army as 1st Lieut., E. O. R. C. and after April 1918 Capt., Engrs., N. A., July 1918 to Aug 1919 in France, on railroad work, being Terminal Road Master, Div. Engr., and Engr., M. of W., and finally Representative of Gen. Supt., 14th Grand (Ry.) Div., handling transfer of Trans. Corps facilities, railway, yards, shops, water service, barracks, camps, etc. *TT 1.6: P 1.6: RC 1.6: D 1.6.*—Aug 1919 to April 1923 not on engineering work—April 1923 to Aug. 1928 Asst. to Chf. Engr., and Aug. 1928 to date Chf. Engr., S. P. of Mex., in direct charge of maintenance of way, structures, water and local service and construction of permanent bridges, facilities, etc.; construction included six major steel and concrete river bridges ($1 188 270) and minor bridges ($1 388 000). *TT 8.8: P 8.8: RC 8.8: D 8.8.*—*TT 24.6: SP 0.2: P 24.4: RC 16.6: D 13.* Refers to E. H. Barkmann, E. H. Connor, W. B. Gregory, C. R. Harding, W. H. Kirkbride, J. C. McClure, W. L. Wilson.

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(11) CRAIG, BURT LEO, Assoc. M., 130 South Pico Ave., Long Beach, Cal. (Elected March 10, 1930.) (Age 38. Born Springfield, Mo.) May 1915 to May 1917 Draftsman, Plains Iron Works Co., Denver, Colo., drafting and tracing small machine parts. *TT 1: SP 1.*—May to Nov. 1917 Draftsman on design of concentrating mill, and Sept. 1921 to April 1924 Structural and Mech. Engr., designing steel and concrete ore bins, conveyor systems, aerial tramways, etc., with U. S. Smelting, Refining & Mfg. Co., Salt Lake City, Utah, and (1 1/6 years) Pachuca, Old Mexico. *TT 2.9: SP 0.3: P 2.6: RC 2.6: D 2.6.*—Nov. 1917 to March 1918 Draftsman, James J. Burke & Co., Salt Lake City, steel detailing and drafting on Steffen's plant for sugar refinery. *TT 0.3: P 0.3.*—March 1918 to Jan. 1919 Private (2 months) and 2d Lieut., Engrs., U. S. Army, in United States, temporary bridges and field fortifications. *TT 0.7: SP 0.2: P 0.5: RC 0.5.*—Jan. 1918 to Aug. 1919 and March 1920 to Jan. 1921 Structural and Mech. Engr., successively with Anaconda (Mont.) Copper Min. Co., on layout and design of crushing plant, and Amalgamated Sugar Co., Ogden, Utah, on layout and design of beet-sugar factories. *TT 1.4: P 1.4: RC 1.4: D 1.4.*—April 1924 to Sept. 1925 Structural Engr. with John M. Cooper, Archt. and Engr., Los Angeles, Cal., designing reinforced concrete, office and loft buildings. *TT 1.5: P 1.5: RC 1.5: D 1.5.*—Sept. 1925 to Feb. 1926 Supt. of Constr. with Walker & Eisen, Archts., Los Angeles, on reinforced concrete buildings. *TT 0.4: P 0.4: RC 0.4.*—Feb. 1926 to Sept. 1927 Structural Engr., successively with Meyer & Holler, Archts. and Engrs., and William Mellema, Archt. and Constr. Engr., both of Los Angeles, designing reinforced concrete office buildings and a warehouse. *TT 1.6: P 1.6: RC 1.6: D 1.6.*—Sept. 1927 to July 1928 Structural Engr., Los Angeles Harbor Dept. at San Pedro, Cal., on design, and July, 1928 to date Chf. Structural Engr., Long Beach (Cal.) Harbor Dept., on design and construction, of wharves, sheds, bulkheads, etc. *TT 4.4: P 4.4: RC 3.6: D 4.4.*—*TT 14.2: SP 1.5: P 12.7: RC 11.6: D 11.5.* Refers to P. Bauman, E. C. Earle, C. T. Leeds, R. G. McGlone, R. E. Marvin, W. Mellema.

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(14) EMANUEL, MORRIS CABLE, Assoc. M., 801 Louderman Bldg., St. Louis, Mo. (Elected Sept. 3 ,1913.) (Age 50. Born St. Louis, Mo.) 1906 B. S. in C. E., Washington Univ. *TT 4: P 4.*—June 1906 to Jan. 1908 on Engr. Corps, Missouri, Pacific & Iron Mountain & Southern Ry., several months in charge of revetment and track protection work, also acting Asst. Div. Engr., Omaha Div. *TT 1.3: SP 0.2: P 1.1: RC 0.6.*—Jan. 1908 to Oct. 1909 Surveyor and Computer, U. S. Board of Examination and Survey of Mississippi River and Mississippi River Comm., computing surveys, studying backwater functions, designing and estimating costs of submerged weirs. *TT 1.7: P 1.7: RC 0.7: D 0.7.*—Oct. 1909 to April 1910 Engr., St. Louis Street Dept., on surveys, establishing lines and grades, estimating and supervising construction of streets and alleys. *TT 0.5: P 0.5: RC 0.5: D 0.5.*—April to Oct. 1910 Supt. of Constr., St. Louis Board of Education, supervised construction on three large schools and designed foundations for one building. *TT 0.5: P 0.5: RC 0.5: D 0.1.*—Oct. 1910 to March 1912 Asst. Engr., Bemis Bros. Bag Co., had charge of construction of addition to Kansas City Bldg. ($96 000) and of a boiler house ($16 000), worked on design and supervised construction of plant in St. Louis ($75 000), also miscellaneous design. *TT 1.4: P 1.4: RC 1.4: D 0.7.*—March 1912 to Jan. 1913 Civ. Engr., Distribution Dept., St. Louis Water Dept., in charge of pitometer survey of city, pipe laying, construction of three substations, etc. *TT 0.8: P 0.8: RC 0.8: D 0.2.*—Jan. 1913 to Sept. 1918 Res. Engr. and Supt. of Constr. with Wm. B. Ittner, Archt., on construction of Ft. Smith (Ark.) High School ($237 000); acted in consulting capacity to Ft. Smith School Board, making changes in plans, etc.; Supt. of Constr., Academy High School (over $1 000 000), Erie, Pa., supervised erection of two grade schools and East Side High School (in part), etc.; spare time as Cons. Engr., also designed Modern Tool Co.'s Plant ($137 000), additions to Erie Brewing Co., detailed steel for war work shop of Erie Forge Co., designed structural work for Kraus Stores, Nagle Boiler Works, etc. *TT 5.7: P 5.7: RC 5.7: D 3.*—Sept. 1918 to Nov. 1920 Senior Plant Engr., U. S. Shipping Board, Emergency Fleet Corporation, Div. of Shipyard Plants, Jacksonville, Fla., being Res. Engr. at South Jacksonville Yards (three) and later also at U. S. Govt. Concrete Yard and Jacksonville Dry Dock & Marine Ry. Yard, in charge for Govt. of all plant work, appraisals, estimates, etc., also designed oil-burner system, concrete storage-tanks and condenser for power plant at Merrill Stevens Shipbuilding Corporation, South Jacksonville, suggested methods and assisted in making side launching tests. *TT 2.2: P 2.2: RC 2.2: D 1.*—Nov. 1920 to Nov. 1921 Res. Engr., Lee St. Viaduct for Hillier Constr. Co., Jacksonville, designed forms, estimated and detailed reinforcing steel, had charge of lines and grades. *TT 1: P 1: RC 1: D 0.5.*—March 1922 to Jan. 1926 with Bldg. Div., Dept. of Public Safety, City of St. Louis, until 1925 as Senior Civ. Engr., then Chf. Engr., checking structural designs and plans

and in charge of inspection of buildings; spare time designed a number of structures, including a 485-ft. cableway (with steel towers) for The Union Quarry & Constr. Co., 2-story reinforced concrete foundry for Green Foundry Co., 2-story flat slab garage in Webster Groves, Mo., steel supports for large steel smoke-stack, etc. *TT 3.8: P 3.8: RC 3.8: D 1.*—Jan. 1926 to date Engr., James Black Masonry & Contr. Co., Engrs. and Contrs., St. Louis, Mo., designing, estimating and supervising construction; designed several buildings for Liggett & Meyers Tobacco Co. (one, mill type, under construction), made studies for strengthening building for Johnson, Stevens & Shinkle Shoe Co., made preliminary design for a large reinforced concrete warehouse, and had charge of structural design of a warehouse ($1 000 000) for International Shoe Co. *TT 6: P 6: RC 6: D 3.*—*TT 29: SP 0.2: P 28.8: RC 23.2: D 10.8.* Refers to B. L. Brown, E. C. Dicke, B. R. Leffler, W. E. Rolfe, E. O. Sweetser, J. L. Van Ornum, E. E. Wall.

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(1) **FLEMING, ERIC**, Assoc. M., 209 Townsend St., New Brunswick, N. J. (Elected Junior, May 28, 1923; Assoc. M., Dec. 15, 1924.) (Age 35. Born New York City.) Registered Prof. Engr., Civ. Engr., Land Surveyor and Archt., State of New Jersey. 1920 B. S. in C. E., and 1923 C. E., Rutgers Univ. *TT 4: P 4.*—June 1915 to June 1920 (while student) Asst. to Asher Atkinson, Civ. Engr., on construction of filtration plant, New Brunswick, N. J., computations, mapping, subdivisions, streets, sewers, being Draftsman and Chf. of Party.—June 1920 to June 1923 Asst. City Engr., New Brunswick, paving ($625 000) and sewers ($47 000), completed City Survey and Tax Map, designed municipal improvements, bridges and highways, in charge of surveys, industrial and housing developments, and railroad construction. *TT 3: P 3: RC 2.7: D 1.2.*—June 1923 to Oct. 1924 Archt. Engr., Elec. Bond & Share Co., New York City, acting as Squad Leader, designing and supervising steam and hydro-electric power-plant design, including Deepwater, Houston, Tex. ($10 000 000) and eleven other plants. *TT 1.2: P 1.2: RC 0.9: D 0.7.*—Oct. to Nov. 1924 Asst. to Designing Engr., Port of New York Authority, made preliminary drawings, land appraisal, estimates for Inland Terminal. *TT 0.1: P 0.1: D 0.1.*—Nov. to Dec. 1924 Appraisal Engr., Sanderson & Porter, Cons. Engrs., New York City, on estimate, appraisal and field survey of New York Edison Co.'s buildings. *TT 0.1: P 0.1: RC 0.1.*—Dec. 1924 to June 1926 Asst. Engr., New York & New Jersey Interstate Bridge & Tunnel Comms., being Squad Boss on drawings and design of four ventilation buildings, Holland Tunnel. *TT 1.5: P 1.5: RC 1: D 0.5.*—June 1926 to Jan. 1927 Archt. with William Whitehill, Archt., New York City, designed Tremont Office Bldg., New York Edison Co. and had charge of plans for other offices and substations in New York City. *TT 0.6: P 0.6: RC 0.6: D 0.6.*—Jan. to March 1927 Designing Draftsman, Equity Constr. Co., Contrs. & Engrs., New York City, on structural and architectural design of plants (reinforced concrete, steel and mill construction) of Ward Baking Co., Birmingham, Ala. and Cambridge plants. *TT 0.2: P 0.2: D 0.1.*—March 1927 to June 1928 and Jan. to July 1929 Archt. and Engr., Isaac Beers Co., Inc., and Structural Engr. and Archt., successively for National Biscuit Co., Watson Eng. Co., and Henry Manley, Cons. Engr., all of New York City, designing, estimating, superintending, etc., on various office buildings, plants, etc., (flat slab, beam and girder, mill and heavy timber and skeleton steel construction), including Daily News and National Biscuit Bldgs., New York City, plans for Atlanta (Ga.) plant ($200 000), and foundations and roof trusses for Cambridge plant, of N. B. Co., additions to plants of Darling (Pa.) Valve Co. ($250 000) and Carlisle (Pa.) Tire & Rubber Co. ($150 000), design of Proctor and Gamble Plant, Baltimore, Md. ($1 000 000), etc. (given in detail in application). *TT 1.7: P 1.7: RC 1.3: D 1.5.*—June 1928 to Nov. 1929 Archt. in Newark, N. J., until Jan. 1929 with Guilbert & Betelle, Archts., on various schools and institutions, including Jersey City Normal School, New Jersey Home for Feeble Minded, Trenton, etc., then with J. H. & W. C. Ely, Archts., plans, design, detail and checking on National Newark & Essex Bank Bldg. ($7 000 000) and others. *TT 0.9: P 0.9: RC 0.2: D 0.6.*—Nov. 1929 to July 1931 Structural Engr. & Archt., until March 1930 with The Austin Co., Engrs. & Contrs., then with Schneider, Kleeman & Werther, Archts., both of Newark, structural steel, reinforced concrete and architectural design, layout, detail, estimates, superintendence, etc. on various schools and commercial and industrial buildings (over $1 400 000). *TT 1.8: P 1.8: RC 1.2: D 1.6.*—Oct. to Dec. 1931 Structural Designer and Asst. to Chf. Engr., Pittsburgh & Erie Coal & Coke Co., Erie, Pa., design of coal-handling equipment. *TT 0.1: P 0.1: RC 0.1: D 0.1.*—Dec. 1931 to date in private practice as Archt. and Engr. *TT 0.3: P 0.3: RC 0.3: D 0.3.*—*TT 15.5: P 15.5: RC 8.4: D 7.3.* Refers to A. Atkinson, H. N. Lendall, H. Manley, Jr., E. D. Powers, E. H. Rockwell, O. Singstad, R. Smillie, C. D. Watson.

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(8) **GRIFFENHAGEN, EDWIN OSCAR,** Assoc. M., 221 North La Salle St., Chicago, Ill. (Elected Oct. 31, 1911.) (Age 45. Born Chicago, Ill.) 1906 B. S., and 1909 C. E., Armour Inst. Tech. *TT 4: P 4.*—1906 (5 months) Eng. Asst. with Arthur Gibson, Nome, Alaska, surveying, drafting and mining engineering. *TT 0.4: P 0.4.*—Jan. 1907 to March 1910 Office Engr., Chicago, Milwaukee & St. Paul Ry., in charge designers and draftsmen on reinforced concrete. *TT 3.2: P 3.2: RC 2.2: D 3.2.*—March 1910 to July 1911 with City of Chicago, until July 1910 as Archt. Engr., Bldg. Dept., responsible for approval of plans for building permits, then Chf. of Technical Staff, Efficiency Div., on reorganization of Dept. of Public Works, etc. *TT 1.3: P 1.3: RC 1.3: D 0.3.*—July 1911 to Jan. 1920 Director of Management, Eng. Dept., Arthur Young & Co., consultants in management and in engineering-economic projects, in complete charge of all engineering work. *TT 8.5: P 8.5: RC 8.5.*—Jan. 1920 to date Senior Partner, Griffenhagen & Associates, Chicago, directing all activities, including studies and reports in connection with large engineering and organization projects, bridges, port developments, etc. *TT 12: P 12: RC 12.*—*TT 29.2: SP 0.2: P 29: RC 24: D 3.5.* Refers to P. Hansen, C. F. Loweth, E. S. Nethercut, J. H. Prior, G. T. Seabury.

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(1) **KRUSE, PAUL FREDERICK,** Assoc. M., 52 William St., New York City. (Elected Aug. 30, 1926.) (Age 40. Born Ogdensburg, N. Y.) 1913 B. S. C. E., Univ. of Vt. *TT 4: P 4.*—Summer 1912 Topographer, Grand Trunk Pacific R.R.—June 1913 to Oct. 1914 with Eng. Dept., Ontario Power Co., Niagara Falls, Ont., design and construction, Instrumentman on construction and Concrete Inspector during installation of two 13 500-h.p. hydro-electric units. *TT 1.2: P 1.2.*—Oct. 1914 to March 1917 Engr. and Constr. Supt. for A. D. Taylor, Landscape Archt., Cleveland, Ohio, on landscape development, including construction of roads, walks, pools, retaining walls, etc., also office design. *TT 2.4: P 2.4: RC 1.5: D 1.*—April to Dec. 1917 Asst. Plant Engr., Norton Co., Chippewa (Ont.) Plant, supervising construction and maintenance of plant and machinery. *TT 0.7: P 0.7: RC 0.5.*—Dec. 1917 to May 1918 with Eng. Dept., National Aniline and Chemical Co., Buffalo, N. Y., plant engineering and appraisal work. *TT 0.4: P 0.4.*—May 1918 to Feb. 1923 Asst. to Hydr. Engr., Niagara Falls (N. Y.) Power Co., plant design and in charge of hydraulic research and tests; also associated with Norman R. Gibson in efficiency test work for other power companies. *TT 4.8: P 4.8: RC 3: D 3.*—Feb. 1923 to date with Sanderson & Porter, Cons. Engrs., New York City, in charge of hydraulic investigation and supervision of design of hydro-electric power developments, also water-power and other engineering investigations and reports; 1930 and 1931 also Lecturer, Polytechnic Inst. of Brooklyn, and in charge of engineering graduates evening course in Water-Power Eng. *TT 9: P 9: RC 9: D 9.*—*TT 22.5: P 22.5: RC 14: D 13.* Refers to F. Blossom, N. R. Gibson, H. P. Hammond, J. P. Hogan, C. W. Larner, F. W. Scheidenhelm, B. E. White.

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(4) **RENZ, EDWIN WILBERT,** Assoc. M., 5024 Osage Ave., Philadelphia, Pa. (Elected Aug. 9, 1920.) (Age 46. Born Philadelphia, Pa.) 1911 B. C. E., Univ. of Mich. *TT 4: P 4.*—Jan. 1906 to Jan. 1907 Draftsman, Elec. Storage Battery Co., battery layouts, substations of electric railways. *TT 0.5: SP 0.5.*—Jan. to Oct. 1907 and Oct. 1911 to Sept. 1912 Draftsman, The Harrison Safety Boiler Works, Philadelphia, Pa., on design of feed water heaters, steam separators and power-plant equipment. *TT 0.9: SP 0.9.*—Sept. 1912 to Jan. 1914 Draftsman, The American Bridge Co., Pencoyd, Pa., steel detailing. *TT 0.7: SP 0.7.*—Jan. 1914 to Sept. 1916 Structural Designer until June 1915 with Frank N. Kneas, Cons. Engr., Philadelphia, then with Trussed Concrete Steel Co., Philadelphia, on design of reinforced concrete office buildings, hotels, power and water-purifying plants, schools, etc. *TT 2.8: P 2.8: D 2.8.*—Sept. 1916 to Nov. 1917 with Warren-Moore & Co., Gen. Contrs., Philadelphia, on design, estimating and inspection of steel and reinforced concrete buildings. *TT 1.2: P 1.2: RC 1.2: D 1.2.*—Nov. 1917 to date with Philadelphia and Newark Offices, The Austin Co., Industrial Engrs. and Bldrs., as Structural Engr., Chf. Draftsman and Philadelphia Dist. Engr., consulted with owners, plant engineers and superintendents, studied plant operations, made surveys, layouts and reports; designed, estimated and supervised design and construction of large industrial and commercial buildings and airports. *TT 14.1: P 14.1: RC 9: D 14.1.*—*TT 24.1: SP 2: P 22.1: RC 10.2: D 18.1.* Refers to H. M. Chapin, W. Dyer, H. C. Gardner, W. H. Gravell, W. S. Lohr, H. F. Miter, J. H. Wickersham.

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(1) **SNYDER, HOWARD HALSEY,** Assoc. M., 183 Madison Ave., New York City. (Elected Oct. 2, 1922.) (Age 41. Born New Rochelle, N. Y.) 1913 C. E., Cornell Univ. *TT 4:*

P 4.—Dec. 1913 to Jan. 1920 with Richard Carvel Co., Inc., until March 1915 as Asst. Engr. on construction, Routes 19 and 22, Sec. 1, New York subway (approx. 1 1/2 miles, $2 500 000), devised and supervised mixing, transportation and placing of concrete, also had direct charge of work connected with maintenance and construction of sub-surface water and gas pipes and electric conduits; March 1915 to March 1918 Prin. Asst. Engr. (under L. A. Ball, Chf. Engr.), on same work, had charge of field forces, designed and had charge of many special features for removal of excavation from main headings and placing of steel and concrete, including disposition of many miles of subsurface water and gas pipes and electric conduit lines and inspection and maintenance of adjoining buildings; after March 1918 Acting Chf. Engr. in charge of completion of same section of subway. *TT 6: P 6: RC 5.2: D 5.2.*—Jan. 1920 to date member of firm, Ball & Snyder, Cons. Engrs., designing and supervising design and construction of structural steel and concrete buildings and foundations for apartment houses, hotels, theatres, loft and office buildings, etc. (over $100 000 000), including Chanin Bldg. (56 stories), San Remo Tower Bldg. (34 stories), Hotel Lincoln (30 stories), Midway Theatre and Hotel (26 stories), Majestic Apartment House (32 stories), all in New York City, Riker's Island (N. Y.) Penitentiary, etc. *TT 12: P 12: RC 12: D 12.*—*TT 22: P 22: RC 17.2: D 17.2.* Refers to L. A. Ball, A. E. Clark, S. F. Holtzman, R. Ridgway, H. V. Spurr, A. N. Van Vleck.

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(7) STARKWEATHER, WALTER HALLETT, Assoc. M., 11645 Pinehurst Ave., Detroit, Mich. (Elected Aug. 26, 1929.) (Age 42. Born Milwaukee, Wis.) Certificate, Univ. de Besancon, France. 1912 Ph.B. in Civ. Eng., Sheffield Sci. School, Yale Univ. *TT 4: P 4.*—June 1912 to Nov. 1917 Rodman, Draftsman, Instrumentman, Chf. of Party, Asst. Engr. and Engr. in Chg. of Constr. with Albert B. Hill, Cons. Engr., New Haven, Conn., on municipal, sanitary, water-supply engineering and construction, etc. *TT 3.8: SP 1.5: P 2.3: RC 1.9: D 0.4.*—Nov. 1917 to May 1920 Private to Master Engr., Senior Grade, 26th Regt., U. S. Engrs. (Water Supply), Office of Chf. Engr., A. E. F. (1½ years), mapping and designing water and sewer systems, Hospital 52, Rimaucourt; Prin. Asst., administration of water-supply material, A. E. F. *TT 1.9: SP 0.6: P 1.3: RC 0.9: D 0.2.*—May to Nov. 1920 Civ. Engr. Special Agt., U. S. Census of Drainage, U. S. Bureau of Census, Washington, D. C., completing census of drainage of agricultural lands in northeastern Indiana and northwestern Ohio. *TT 0.5: P 0.5: RC 0.5.*—Dec. 1920 to May 1921 Camp Draftsman, Interstate R R. Co., Norton, Va., on location, mapping, etc. *TT 0.3: SP 0.2: P 0.1: D 0.1.*—June 1921 to June 1922 Instructor (night courses) in Civ. Eng., George Washington Univ., Washington, D. C.; also in private practice of civil engineering, drafting, etc. *TT 1: P 1: RC 0.5: D 0.1.*—July 1922 to May 1926 Dist. Constr. Engr., Ninth Dist., U. S. Coast Guard, Washington, D. C., at Detroit, Mich., in charge of construction and repair work for life-saving stations on Lakes Ontario, Erie and Huron and (part of time) on all Great Lakes, preliminary surveys, reports, estimates, designs, purchases and supervision. *TT 3.7: P 3.7: RC 3.7: D 2.*—May 1926 to date with City Engr.'s Office, Dept. of Public Works, Detroit, Mich., until July 1926 as Chf. of Field Survey Party, on general municipal engineering work, and since July 1926 Engr. of Surveys, in charge of Survey Div. and Topographical Office, being responsible for field engineering on surveys, construction, also for special assignments, investigations and reports, construction included sewers ($42 000 000) and paving ($44 000 000). *TT 5.8: P 5.8: RC 5.5:*—*TT 20.9: SP 2.2: P 18.7: RC 13: D 2.8.* Refers to C. M. Blair, C. Y. Dixon, P. A. Fellows, J. P. Hallihan, F. G. Ray, J. W. Reid.

FROM THE GRADE OF JUNIOR

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(2) ABBOTT, RUSSELL WARD, Jun., 36 Trowbridge St., Cambridge, Mass. (Elected Oct. 10, 1927.) (Age 28. Born Tecumseh, Mich.) Registered Civ. Engr., State of Michigan.—1925 A. B., Albion Coll. 1927 B. S. E. (C. E.), Univ. of Mich. *TT 4: P 4.*—Sept. 1925 to Sept. 1926 Commercial Engr., Commercial Eng. Dept., Michigan Bell Telephone Co., Detroit, on future population studies of Michigan cities, after Sept. 1926 being Supervisor. *TT 0.6: SP 0.4: P 0.2.*—June 1927 to Feb. 1929 with City of Toledo, Ohio, until Dec. 1927 as Draftsman, Eng. Dept., Paving & Storm Water Sewer Div., on street and sewer layout, Dec. 1927 to Nov. 1928 Draftsman, Designer and Engr., Bridge Div., on steel and concrete design and drafting, and after Nov. 1928 Engr., on extensive repairs to double-leaf bascule bridge over Maumee River. *TT 1.3: SP 0.2: P 1.1.: RC 0.2.*—Feb. to April 1929 Detailer, Mt. Vernon (Ohio) Bridge Co., detailing steel structures. *TT 0.1: SP 0.1.*—April 1929 to Nov. 1930 Designing Draftsman, Dept. of Archt., Board of Education, Toledo, responsible design in steel and concrete for school buildings. *TT 1.6:*

P 1.6: RC 1.6: D 1.6.—Nov. 1930 to Sept. 1931 Designing Draftsman, with Hoad, Decker, Shoecraft & Drury, Cons. Civ. Engrs., Ann Arbor, Mich., responsible design in steel and concrete for sewage and water-supply structures. *TT 0.8: P 0.8: RC 0.8: D 0.8.*—Sept. 1931 to date graduate student, Dept. of Civ. Eng., Massachusetts Inst. of Technology, Cambridge, Mass. *TT 8.4: SP 0.7: P 7.7: RC 2.6: D 2.4.* Refers to A. J. Decker, A. S. Forster, C. M. Fyler, S. D. Porter, R. H. Sherlock, G. A. Taylor.

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(1) ALPERIN, MAX, Jun., 1674 Bryant Ave., New York City . (Elected April 15, 1929.) (Age 29. Born Ustilug, Poland.) Licensed Prof. Engr., State of New York.—1924 B. S. in C. E., Cooper Union Night School of Sci. 1928 C. E., Brooklyn Pol. Inst. *TT 4: P 4.* —Has completed requirements for M. A. in Math., Columbia Univ.—April 1923 to Oct. 1925 with Board of Water Supply as Axeman, running transit and level, etc. and Rodman assisting on tests, estimates, computations, etc. *TT 0.6: SP 0.6.*—Oct. 1925 to Jan. 1926 Transitman and Computer, Bureau of Highways, Brooklyn, N. Y., designing reinforced concrete footings, slabs and girders, computations, estimates, etc. *TT 0.3: P 0.3: D 0.3.*—Jan. 1926 to Oct. 1930 Jun. Engr., Broad of Transportation, until Jan. 1928 in responsible charge of field work, Route 105, Sec. 2 of new Washington Heights Subway System, investigated and studied soil conditions, etc., supervised computation of estimates, after Jan. 1928 had charge of final estimate and directed construction of subway section, Route 105, Sec. 3 and of foundations and superstructures of buildings and shops in new 207th St. yard. *TT 4.8: P 4.8; RC 4.8.*—Oct. 1930 to date Asst. Engr., Dept. of Finance, New York City, acting as Comptroller's Representative, supervised construction and installation of mechanical equipment, in accordance with plans and specification of various public buildings in Borough of Bronx, makes special investigations and tests of mechanical systems, etc. for construction of various public buildings and prepares reports for Comptroller. *TT 1.3: P 1.3: RC 1.3.*—*TT 11: SP 0.6: P 10.4: RC 6.1: D 0.3.* Refers to B. J. Ahearn, R. W. Armstrong, E. G. Haines, A. Lyle, J. D. Schwartz, S. Weinman.

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(6) BROWN, ALBERT ABRAHAM, Jun., 5735 Hobart St., Pittsburgh, Pa. (Elected July 11, 1927.) (Age 32. Born New York City.) 1925 B. S., Carnegie Inst. Tech. *TT 4: P 4.*—Aug. 1918 to Aug. 1921 Asst. Chemist, Fiske Bros. Refining Co., Newark, N. J. *TT 1.5: SP 1.5.*—Aug. 1925 to Aug. 1926 Estimator and Detailer, and March 1927 to May 1928 Designer and Checker, of structural steel, Jones & Laughlin Steel Corporation, Keystone Works, Pittsburgh, Pa. *TT 1.7: SP 0.5: P 1.2: D 1.2.*—Aug. 1926 to March 1927 Structural Detailer and Checker, Lukens Steel Co., New Orleans, La. *TT 0.3: SP 0.3.*—May 1928 to date Structural Engr., H. H. Robertson Co., Pittsburgh, Pa. on preliminary designs and estimates for industrial buildings, in charge of loading tests on corrugated steel sheets for Goodyear Zeppelin Hangar at Akron, studies of corrugated steel sheets under various loading conditions, about 1¼ years in charge of engineering data on Keystone steel flooring; designing and revising designs of buildings to suit steel flooring, studying highway-bridge floors, methods of reducing dead loads, etc. *TT 3.7: P 3.7: RC 1.2: D 3.7.*—*TT 11.3: SP 2.3: P 9: RC 1.2: D 3.7.* Refers to C. G. Dunnells, F. H. Frankland, B. F. Hastings, F. Kubitz, J. R. Sexton, C. B. Stanton.

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(8) CORCORAN, LOUIS PAUL, Jun., 33 West Grand Ave., Chicago, Ill. (Elected April 18, 1927.) (Age 29. Born Wallace, Idaho.) 1926 B. S. in Mech. Eng., Wash. State Coll. *TT 4: P 4.*—Sept. 1922 to Sept. 1923 Instrumentman, Pacific Telephone & Telegraph Co, Spokane, Wash. *TT 0.5: SP 0.5.*—June to Oct. 1925 Asst. City Engr., Pullman, Wash., on erection of sewage-disposal plant.—Sept. 1926 to date with Portland Cement Association, until Jan. 1928 as Asst. Field Engr., studied scientific design of concrete in Research Laboratory, inspected concrete pavement construction, etc., then Office Engr., Chicago, giving technical information on use of Portland cement, and after June 1929 Field Engr., inspecting concrete construction, preparing and revising specifications, trouble shooting on defective work, etc. *TT 4.6: SP 0.7: P 3.9: RC 2.1.*—*TT 9.1: SP 1.2: P 7.9: RC 2.1.* Refers to A. W. Consoer, E. M. Fleming, H. F. Gonnerman, F. R. McMillan, U. F. Turpin, G. E. Warren, F. A. Windes.

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(14) CROFOOT, DAVID WILSON, Jun., Jefferson City, Mo. (Elected Nov. 11, 1929.) (Age 30. Born Vandling, Pa.) 1923 A. B., and Sept. 1923 to June 1924 graduate student in Civ. Eng., Cornell Univ. *TT 3: P 3.*—June 1924 to Sept. 1925 Inspector, Pennsylvania

Dept. of Highways, inspecting and computing grading, drainage structures and concrete pavement. *TT 0.6: SP 0.6.*—Sept. 1925 to Oct. 1929 with Alabama Highway Dept., until Aug. 1926 as Transitman on highway location (30 miles) and on concrete pavement (7 miles), then Res. Engr. in charge of location and construction of state highways, including sand clay, top soil, gravel and concrete pavement, timber and concrete bridges. *TT 3.6: SP 0.4: P 3.2: RC 3.2.*—Oct. 1929 to date with Missouri Highway Dept., until Oct. 1930 as Designer, checking and preparing highway plans, making up proposals, writing special provisions and making estimates of cost, and since Oct. 1930 Project Engr., in charge of construction of state highways. *TT 2.4: P 2.4: RC 2.4: D 1.*—*TT 9.6: SP 1: P 8.6: RC 5.6: D 1.* Refers to R. W. Brooks, C. W. Brown, S. H. Clelland, W. Finnell, W. O. Hill, C. R. Hopper, H. H. Houk, G. L. Moulton.

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(2) **FARWELL, HERBERT FREEMAN,** Jun., 88 Sycamore St., Roslindale, Mass. (Elected May 19, 1924.) (Age 32. Born Roslindale, Mass.) 1923 B. S. in C. E., Tufts Coll. *TT 4: P 4.*—March to Nov. 1923 Rodman and Draftsman, Boston & Maine R. R., survey and mapping of pile-trestle work at North Station, Boston, Mass. *TT 0.4: SP 0.4.*—Nov. 1923 to Nov. 1924 Transitman and Draftsman, Worcester County Highway Dept., instrument work on right-of-way location, office computation and drafting. *TT 0.5: SP 0.5.*—Nov. 1924 to March 1925 Chf. of Party with R. S. Baylis, Civ. Engr., surveys, computations and mapping. *TT 0.3: P 0.3.*—March 1925 to March 1931 with Turner Constr. Co., until May 1927 as Timekeeper and Job Accountant, then Line and Gradesman, laying out excavations and building foundations and checking lines and grades during construction, and after Sept. 1929 Asst. Supt. in charge of field engineering during construction and supervising sub-contract work. *TT 4.9: SP 1.1: P 3.8: RC 2.4.*—March 1931 to date Asst. Supt., Kalman Floor Co., Watertown, Mass., in charge of installation of granolithic floors. *TT 0.9: P 0.9: RC 0.9.*—*TT 11: SP 2: P 9: RC 3.3.* Refers to H. P. Burden, B. W. Guppy, L. O. Marden, W W. Roberts, Jr., C. H. Schwertner, E. H. Wright.

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(10) **FEAGIN, LAWRENCE BAKER,** Jun., Volunteer Bldg., Chattanooga, Tenn. (Elected Dec. 15, 1924.) (Age 32. Born Montgomery, Ala.) 1922 A. B., Vanderbilt Univ.—1924 S. B., Mass. Inst. Tech. *TT 4: P 4.*—July 1924 to date with U. S. Engr. Office, at Florence, Ala. and Chattanooga, Tenn., until Dec. 1924 as Draftsman and on power studies, etc.; Dec. 1924 to Feb. 1929 Jun. Engr., being Inspector of control and lighting conduits and electrical installation in switchboard and oil-circuit breaker buildings and Wilson Dam, also on efficiency tests of turbines and generators at same dam (Dec. 1924 to May 1926), Field Inspector on reinforced concrete utility building and high-tension switch-yard (Dec. 1926 to May 1927), Wilson Dam, and power, river-hydraulic and flood-control studies, etc. (May 1927 to July 1928); Feb. 1929 to Sept. 1931 Asst. Engr., having general supervision of maintenance and improvement on Tennessee River from Hales Bar to Riverton, including dike construction, dredging, drafting, maintenance of gauge records and preparing estimates of costs and quantities, reports, economic studies of canalization of Tennessee River, etc.; since Sept. 1931 Engr., being Co-ordinating Eng. Asst. to Dist. Engr., Chattanooga Dist., having general supervision and co-ordination of all work of Dist. *TT 6.9: SP 0.7: P 6.2: RC 3.6.*—*TT 10.9: SP 0.7: P 10.2: RC 3.6.* Refers to P. B. Hill, W. R. King, L. G. Puls, J. C. Stiles, J. Wright, C. A. D. Young.

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(2) **FEER, HUGO ANTHONY,** Jun., 459 Penobscot Ave., Millinocket, Me. (Elected Dec. 15, 1924.) (Age 31. Born Zurich, Switzerland.) 1923 C. E., Ecole Pol. Federale.—Aug. 1922 to Jan. 1923 Sec. Engr., Simonett & Co., supervising in tunnel during construction of penstock for hydro-electric plant. *TT 0.5: P 0.5.*—Aug. 1923 to Jan. 1924 Civ. Engr., Atelier de Construction d'Enghien, on design of steel construction. *TT 0.5: P 0.5: RC 0.5.*—Feb. 1924 to Sept. 1925 Draftsman and Designer, International Paper Co., drafting and design of hydro-electric plants, saw mills and paper mills. *TT 1.6: SP 0.1: P 1.5*—Oct. 1925 to March 1926 Draftsman and Designer, Lockwood Greene & Co., on steel and reinforced concrete for rayon mill. *TT 0.5: P 0.5.*—April to Aug. 1926 abroad.—Aug. to Sept. 1926 Engr., International Paper Co., Fort Edwards mill, on design of pulpwood grinder. *TT 0.1: SP 0.1.*—Oct. 1926 to date Engr., Great Northern Paper Co., design and drafting of buildings and installation of machines for paper and pulp mills. *TT 5.3: P 5.3: RC 2: D 3.*—*TT 8.5: SP 0.1: P 8.4: RC 2.5: D 3.* Refers to O. H. Ammann, F. C. Bowler, E. Hutchins, H. M. Nelson, G. S. Wood.

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(2) **FREEMAN, CASSIUS WILLIAM,** Jun., 336 Brewer St., East Hartford, Conn. (Elected June 6, 1927.) (Age 32. Born East Hartford, Conn.) May 1918 to June 1929 with Buck & Sheldon, Inc., as Rodman, Transitman and Chf. of Party on surveys, sewer systems, highways, building construction, Draftsman on structural and mechanical equipment of buildings, Supervisor and Inspector on construction. *TT 9.2: SP 1.9: P 7.3: RC 1: D 0.5.*—June 1929 to July 1930 with Paul Sheldon, on industrial and commercial buildings, drafting, design and field layout. *TT 1.1: P 1.1: RC 0.2: D 0.4.*—July 1930 to Oct. 1931 Engr. and Asst. Supt., Industrial Constr. Co., on construction of industrial and commercial buildings. *TT 1.3: P 1.3: RC 1.3.*—Oct. to Nov. 1931 Supt. with R. H. Cone, on State Road construction. *TT 0.1: P 0.1: RC 0.1.*—Nov. 1931 to date with E. J. Vaughn, investigating condition and inspecting reconditioning of steel stand-pipe for Metropolitan Dist. Water Bureau. *TT 0.2: P 0.2: RC 0.2.*—*TT 11.9: SP 1.9: P 10: RC 2.8: D 0.9.* Refers to H. R. Buck, W J. Ennis, J. T. Henderson, G. L. Mylchreest, R. S. Topper, P. R. Williamson.

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(11) **IRWIN, LUTHER WESLEY,** Jun., 972 Amherst Pl., Los Angeles, Cal. (Elected June 9, 1930.) (Age 29. Born LaGrange, Ill.) Registered Civ. Engr., State of California. Aug. 1920 to Feb. 1922 student, Coll. of Eng., Univ. of Cincinnati. *TT 0.5: P 0.5.*—May to Aug. 1922, May to Dec. 1923 and May to Sept. 1924 with Arrowhead Lake Co., San Bernardino County, Cal., as Chainman, Transitman and Levelman on sewer, road and waterline location and surveying and Inspector on sanitary sewer construction, stream measurement, computing, etc. *TT 1: SP 0.5: P 0.5.*—Aug. to Dec. 1922 Foreman, Public Works Constr. Co., Cincinnati, Ohio, on construction of 3½ miles of asphalt paving. *TT 0.1: SP 0.1.*—Feb. to May 1923 Chainman, Clarence P. Day Corporation, Pasadena, Cal., and Dec. 1923 to May 1924 Computor with Salisbury, Bradshaw & Taylor (5 months) and George E. Daley (1 month), Los Angeles, on subdivision work. *TT 0.4: SP 0.4.*—Sept. 1924 to July 1925 Draftsman, until Nov. 1924 with A. C. Pillsbury, Beverly Hills, Cal., then with Southern California Telephone Co. *TT 0.4: SP 0.4.*—July 1925 to date Jun. Civ. Engr., Bureau of Eng., Los Angeles, 2 years computing and drafting, Street Opening and Widening Div., then on sanitary sewer design, Venice Dist. Office. *TT 6.5: P 6.5: RC 3.5: D 3.*—*TT 8.8: SP 1.3: P 7.5: RC 3.5: D 3.* Refers to P. Bauman, E. H. Clarkson, Jr., R. L. Derby, J. D. Faulkner, P. Fuller, E. D. Lownes, W. D. Potter, I. F. White.

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(5) **KAUFHOLZ, WILLIAM,** Jun., 1114 North Patterson Park Ave., Baltimore, Md. (Elected June 6, 1927.) (Age 31. Born Baltimore, Md.) 1925 B. E. in C. E., Johns Hopkins Univ. *TT 4: P 4.*—June to Nov. 1918 Levelman, E. L. Scheidenhelm Co., on construction of Camp Holabird, Md. *TT 0.4: SP 0.4*—Feb. 1919 to Feb. 1920 Chainman, Rodman and Topographer, Baltimore & Ohio R. R. *TT 0.5: SP 0.5.*—March to May 1920 Draftsman, Engr. and Foreman, Public Amusement Co., building amusement devices in parks. *TT 0.1: SP 0.1.*—June to Oct. 1920 Draftsman for J. Spence Howard, Civ. & Cons. Engr., Baltimore, Md., map making, real estate subdivision. *TT 0.2: SP 0.2.*—Nov. 1920 to Dec. 1921 Computer, Western Maryland Ry., valuation work, estimating costs of buildings, trackage, etc. *TT 0.6: SP 0.6.*—Summer 1924 Chf. of Party, Sam Diescher & Bros., Cons. Engrs., Pittsburgh, Pa., laying out buildings, sewers, water lines, machinery foundations, etc., figuring for contractors payments on plant for Standard San. Mfg. Co., Baltimore.—Summers 1922 and 1923 and July 1925 to date with Bethlehem Steel Co., Sparrows Point, Md., first as Draftsman, detailing structural steel, foundations, railroad tracks, etc., July 1925 to March 1926 Designing Draftsman on structural steel and reinforced concrete, and since March 1926 Squad Leader, on design and detail of structural steel and concrete for new 1000-ton blast furnaces, mill buildings, highway bridge, underpass, etc. *TT 6.5: P 6.5: RC 5.8: D 6.5.*—*TT 12.3: SP 1.8: P 10.5: RC 5.8: D 6.5.* Refers to T. F. Comber, Jr., E. M. Killough, F. W. Medaugh, C. C. Singleton, J. T. Thompson, A. J. Warlow.

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(16) **KELLY, IRA DAVID SANKEY,** Jun., 1500 Stringham Ave., Topeka, Kans. (Elected July 16, 1928.) (Age 31. Born Chicago, Ill.) 1924 B. S. in Civ. Eng., and 1929 C. E. Kans. State Coll. *TT 4: P 4.*—June 1924 to Feb. 1926 and April 1927 to July 1929 Jun. Highway Engr., Grade IV, Illinois Dept. of Public Works and Buildings, Div. of Highways, Dist. 9, Carbondale, Ill., inspecting construction of pavement, culverts and bridge substructure and since April 1927 engineering and inspection of grading, culverts, pavement and bridges. *TT 3.9: P 3.9: RC 2.2.*—July 1929 to date with Dept. of Design, Kansas Highway Comm., Topeka, Kans., until June 1930 as Squad Chf. on location, survey and plans,

and since June 1930 Designer, designing and checking culvert and bridge plans, special study of welding design. *TT 2.6: P 2.6: RC 2.6: D 2.6.*—*TT 10.5: P 10.5: RC 4.8: D 2.6.* Refers to L. E. Conrad, O. J. Eidmann, F. W. Epps, M. W. Furr, C. H. Scholer.

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(13) **MORRISON, DEMING WILLIAM,** Jun., 1810 T St., Sacramento, Cal. (Elected Dec. 14, 1925.) (Age 29. Born Lexington, Ky.) 1925 A. B. in Civ. Eng., Stanford Univ. *TT 4: P 4.* Nov. 1925 to July 1927 with Fred H. Tibbetts, Civ. Engr., San Francisco, as Transitman and Inspector on location and construction, Nevada County Irrigation Dist. *TT 0.8: SP 0.8.*—Aug. 1927 to March 1930 with Leland S. Rosener, Cons. Engr., San Francisco, until Nov. 1928 as Draftsman on various industrial plant jobs, including design of minor structures, then Res. Engr. on reconstruction of paper mill in Sumner, Wash., installation of machinery, concrete construction, mill building, also construction of 300-ft. timber and pile wharf in Port Angeles, Wash., for Fibreboard Products, Inc., purchased material and after June 1929 Structural Designer, supervised construction of various industrial wooks, estimates and specifications. *TT 2.6: P 2.6: RC 1.4: D 0.8.*—March 1930 to date with Dept. of Dams, State of California, until Feb. 1931 as Jun. Hydr. Engr., then Asst. Hydr. Engr., on analysis of arch, gravity and buttress type dams, including spillway studies, preparation of reports, etc. *TT 1.9: P 1.9: RC 1.9.*—*TT 9.3: SP 0.8: P 8.5: RC 3.3: D 0.8.* Refers to G. W. Hawley, W. L. Huber, E. Hyatt, E. W. Kramer, L. B. Reynolds, L. S. Rosener, R. G. Wadsworth.

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(15) **SHIELS, THOMAS DAVID,** Jun., Box 847, Abilene, Tex. (Elected June 4, 1928.) (Age 31. Born Leonad, Tex.) 1923 B. S. in C. E., Va. Mil. Inst. *TT 4: P 4.*—Sept. 1923 to Aug. 1925 with Florida State Road Dept., until Jan. 1924 as Senior Draftsman, then Instrumentman, April to Oct. 1924 Asst. Project Engr. on concrete bridge construction, Oct. 1924 to May 1925 Project Engr. on 9 miles of road grading and bridge work, at Eau Gallie, Fla. and after May 1925 Asst. on location. *TT 1.4: SP 0.6: P 0.8: RC 0.8.*—Aug. to Sept. 1925 Field Engr., Thomas L. Holland Co., Inc., Miami, Fla., in charge of dredging. *TT 0.1: P 0.1: RC 0.1.*—Nov. 1925 to date Field Engr., Portland Cement Association, promotional, design and field inspection, Dallas (Tex.) Dist. *TT 6.2: P 6.2: RC 6.2: D 3.1.*—*TT 11.7: SP 0.6: P 11.1: RC 7.1: D 3.1.* Refers to F. L. Bramlette, C. A. Clark, C. S. Henning, Jr., D. Lee, J. W. Porter.

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(7) **SMITH, WALDO EDWARD,** Jun., 717 Tenth Ave. North, Fargo, N. Dak. (Elected Nov. 14, 1927.) (Age 31. Born New Hampton, Iowa.) 1923 B. E. in C. E., and 1924 M. S. in C. E., Univ. of Iowa. *TT 4: P 4.*—June 1924 to Jan. 1926 with Burns & McDonnell Eng. Co., Kansas City, Mo., until Aug. 1925 as Office Asst. studying maps, etc., preparing preliminary report on 120 000-h.p. hydro-electric project, drafting, checking, minor design, etc., on water-supply and sewerage projects, then Res. Engr., in charge of construction of reinforced concrete covered reservoir at Ponca City, Okla. *TT 1.3: SP 0.3: P 1: RC 0.6: D 0.2.*—Jan. to Feb. 1926 Instrumentman with Black & Veatch and E. M. Stayton Cons. Engrs., Kansas City, on preliminary sewer work. *TT 0.1: SP 0.1.*—Feb. 1926 to Feb. 1927 and summer 1927 with Black & Veatch, Cons. Engrs., Kansas City, until June 1926 as Asst. Field and Office Engr., then Res. Engr. on steam-power plant improvements at Kingfisher, Okla., including addition to building and installation of boiler, and after Oct. 1926 Asst. Office Engr. *TT 1.3: P 1.3: RC 0.9: D 0.2.*—Feb. 1927 to July 1928 Instructor in Theoretical and Applied Mechanics, Coll. of Eng., Univ. of Illinois, directly responsible to Department Head for conduct of courses. *TT 1.3: P 1.3: RC 1.3.*—July 1928 Collector of field data on sewerage system of Urbana, Ill. *TT 0.1: P 0.1: RC 0.1.*—Aug. 1928 to Aug. 1931 Associate Prof. of Civ. Eng. and Acting Head of Dept. of Civ. Eng., Robert Coll., Constantinople, Turkey, responsible for his own courses and general supervision of other civil engineering courses and for administrative work of department. *TT 3: P 3: RC 3.*—Sept. 1931 to date Asst. Prof. of Civ. Eng., North Dakota State Coll., Fargo, N. Dak., responsible for various civil engineering courses and in charge of some laboratories and equipment and for part-time Asst. *TT 0.4: P 0.4: RC 0.4.*—*TT 11.5: SP 0.4: P 11.1:RC 6.3: D 0.4.* Refers to J. L. Crane, Jr., M. L. Enger, D. C. Jackson, R. E. McDonnell, F. A. Nagler, N. T. Veatch, Jr., S. M. Woodward.

The Board of Direction will consider the applications in this list not less than thirty days after the date of its issue.

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AMERICAN SOCIETY OF CIVIL ENGINEERS

COMING MEETINGS

ANNUAL CONVENTION

YELLOWSTONE NATIONAL PARK

The dates for the Annual Convention have not yet been fixed. As soon as the decision is made and the program formulated, due notice will be given.

The Reading Room of the Society is open from 9:00 A. M. to 5:00 P. M. every day, except Sundays and holidays; from May to September, inclusive, it is closed on Saturdays at 12:00 M.

Members, particularly those from out of town, are cordially invited to use this room on their visits to New York, to have their mail addressed there, and to utilize it as a place for meeting others. There is a file of 274 current periodicals, the latest technical books, and the room is well supplied with writing tables.